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Engineering News-Record

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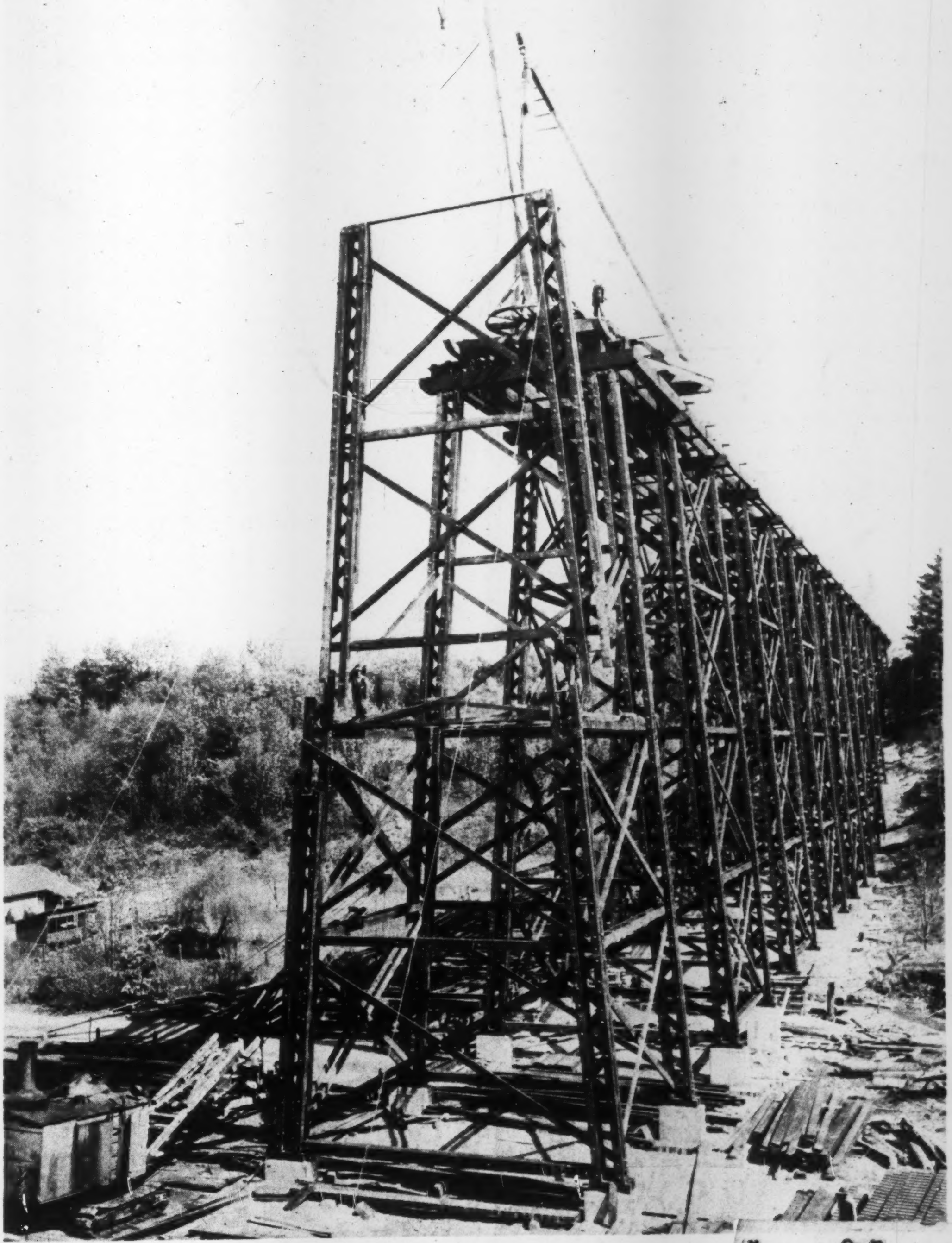

At Port Angeles, Wash., two high timber trestles have bee built employing modern developments in timber constructio

Dredge Plant
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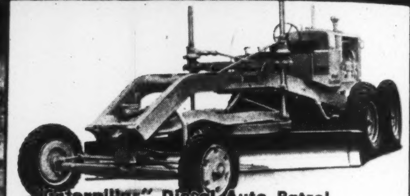
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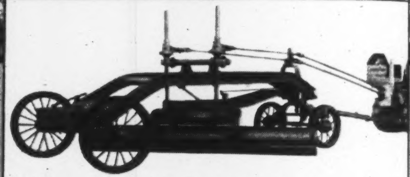
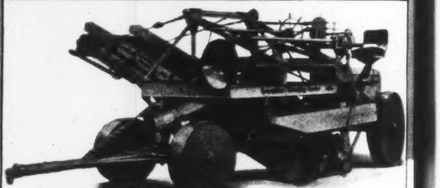
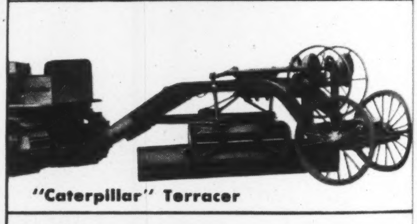

"Caterpillar" Trailer Patrol


"Caterpillar" Elevating Grader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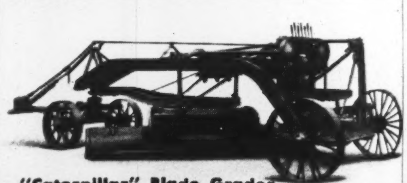
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A MOMENT WITH THE PUBLISHERS

VALUES—*Direct and Indirect*

DURING the ceremonies that recently opened the the Triborough Bridge in New York City, Robert Moses, Chairman of the Triborough Bridge Authority, made one statement that deserves to be noted and remembered in connection with public works policies and projects.

"The Triborough", said the Chairman, "is not just a bridge, nor yet a crossing. It is a great artery, connecting three boroughs of the city, and reaching out at its borders into adjacent counties and states. It's not merely a road for automobiles and trucks—it is a general city improvement * * *."

The interest of this statement lies in its recognition of the fact, so frequently forgotten, that any large public improvement creates values and renders services far beyond the limited circle of those who may use it directly. The same may be said, of course, for any community service facility, whether built by public or private enterprise. There is, however, one important difference.

In the nature of private enterprise, those who make direct use of the service it renders must pay all the costs of that service. If the enterprise is to remain solvent it must eventually take in as much as it spends. These revenues can come only from the direct users—its customers; it is they who must pay all the costs, regardless of what may be the incidental benefits to the rest of the community. That is true of a toll bridge, a power utility or an amusement park; that is the normal procedure of private enterprise.

But in some cases, even of private enterprise, that rule breaks down under the impact of the realities. The subsidy is one device that is used to deal with such cases.

WHEN government paid subsidies to canal and railroad companies it recognized that the public values they would create justified such compensation to the enterprisers over and above what they could collect from their customers. So many who never might use or even see the improvements would benefit from them, that government undertook through taxation to distribute some of the costs over the community as a whole. Otherwise it was possible that the realization of latent community values might be long deferred.

When government pays a subsidy to shipping or air-transport it is but following the same policy that it applied in other days to other forms of transport and for similar reasons. This is no attempt to appraise the wisdom or the honesty with which this procedure may have been applied; it is intended simply to point out that even on public facilities conducted by private enterprise, provision often must be made for revenues in addition to those that can be collected from direct users.

THERE is no reason, therefore, why a principle that private enterprise sometimes must abandon in the case of public facilities should be applied to publicly built and operated facilities. It is true that some forms of facility, the benefits of which can be restricted and measured out to individual citizens, do permit application of the rule that normally governs private enterprise—let the direct users pay the full cost. But unfortunately they are the exception. In the cases of trunk highways, flood control, far-reaching drainage and sanitary works and many other public facilities, no such rule can apply. The benefits are too widespread and the beneficiaries too numerous, obscure or mobile for us to trace down each unit of benefit and collect for it. Only a portion of the total costs can reasonably be charged against the direct users.

All this is no defense of extravagant public spending for facilities that cannot justify themselves in either direct or indirect benefits. Every service rendered by either public or private enterprise must be paid for by someone; if it is not worth what it costs it is an extortion, whether under public or private auspices. This is simply a reminder that the argument, so frequently made, that the direct users of highways and other public facilities should pay all the costs does not recognize a basic difference between private and public enterprise.

Willard Chevalier

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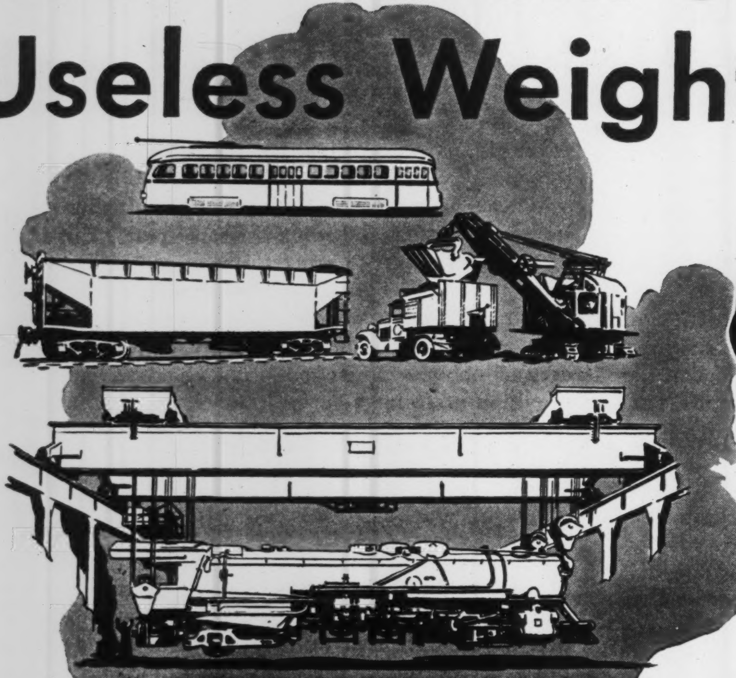
Inland Hi-Steel Solves Your Problem of Useless Weight



● Useless weight is costly. On equipment you use, power is wasted dragging around useless weight. On equipment you may build for sale, useless weight is fast becoming a sales handicap.

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AUGUST 6, 1936

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■

Bridge Wire Failure Analyzed —In 1929, the engineering world was shocked by an alarming number of breakages in a new heat-treated bridge wire in the cables of the Mt. Hope and Detroit suspension bridges. Not yet completed, the bridges were dismantled and rebuilt with conventional cold-drawn wire.

- A six-year investigation of the failed wire by the Bureau of Standards has now thrown important light on these spectacular and mysterious failures. A report of the tests will be published in next week's issue.

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August 6, 1936

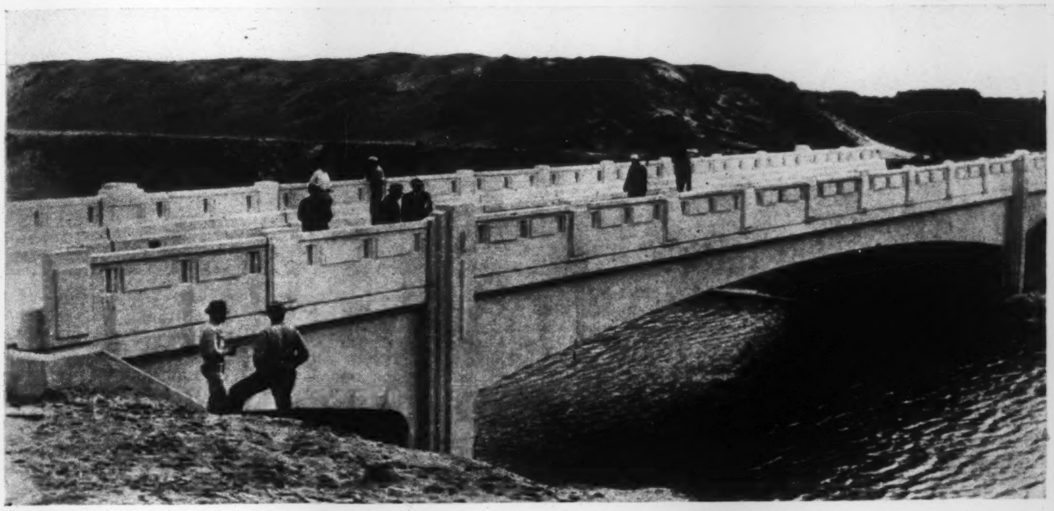


FIG. 1—NEW RECORD SPAN concrete rigid frame bridge in Nebraska. Of ribbed type, its main span is 109 ft. long. The two cantilever side spans are counterweighted.

Fifty-Six Bridges To Span New Power Canal

Longest concrete rigid frame in this country and first with counterweighted cantilever side spans is among structures built over its new canal by the Loup River Public Power District in Nebraska

By Justin Nicolet

Designer, Harza Engineering Co. Columbus, Nebr.

IN ORDER to keep the roads crossing its 33-mile power canal open for traffic, it has been necessary for the Loup River Public Power District in Nebraska to enter upon an ambitious bridge building program. After prolonged preliminary investigations and negotiations with farm owners, city, county, state and federal authorities and with two railroad companies, the district decided upon the construction of twenty-four farm and drainage bridges, twenty-four county bridges, one industrial truss bridge, two railroad bridges, two city street bridges and three state highway bridges. A number of interesting designs were developed, among them some concrete rigid frames of special type, one of which has a 109-ft. span, the longest yet used in this country. The construction program is proceeding rapidly and it is expected that all fifty-six bridges will be completed by August 15.

The Loup River project (ENR Oct. 11, 1934, p. 469) involves the construction of a diversion dam, desilting works, a 33-mile power canal of 3,000 sec.-ft. capacity, and two hydro-electric plants located along the canal near the towns of Monroe and Columbus, Nebr., respectively. The general direction of the canal is west to east. It traverses a very well populated farming territory, with existing county roads one mile apart in either direction, and in addition crosses four state highways. The bridging problem thus was a major one.

Although the majority of the bridges are of timber or of steel construction with concrete decks, the two city bridges and the three state highway bridges are of concrete rigid frame type. Of these the one carrying U.S. highway 30 is most notable, being a ribbed-type frame having a 109-ft. center span and two counter-weighted cantilever side spans of 26½ ft. each. The other four bridges, two in the village of Genoa and one each on U.S. highways 22 and 81 are of two span solid-barrel type with span lengths of from 57 to 60 ft.

Among the major factors that led to the adoption of the concrete rigid frame type of structures may be mentioned:

(1) Minimum obstruction to flow of water in the canal.

(2) Minimum headroom requirements.

(3) Low initial cost with minimum charges for maintenance and replacements.

(4) Adaptability to site and anticipated future heavy traffic.

(5) Maximum utilization of local unskilled and semi-skilled labor.

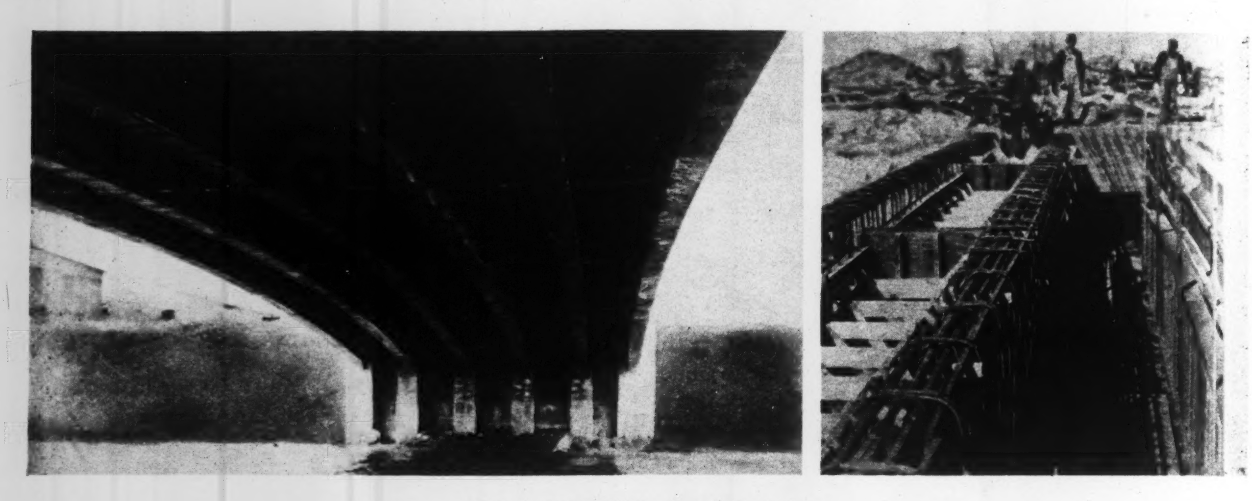


FIG. 2—SIX RIBS, heavily reinforced, furnish the deck support for the new 109-ft. span, concrete rigid frame of the Loupe River Public Power District.

(6) Maximum benefit to community because of use of local materials.

New record span length

The 109-ft. span rigid frame is located three miles east of Columbus at the point where the tailrace canal is crossed by U. S. 30—the Lincoln Highway and the main traffic artery between Omaha and the West. At the bridge site the canal has a width of 100 ft. between edges of beams and 162 ft. at the elevatoin of the roadway. The groundwater level is about 10 ft. above the canal bottom. Since hydraulic considerations demanded an undisturbed and undiminished canal flow section, only a single span crossing structure could be considered.

Insufficient vertical clearances and unsuitable foundation materials, consisting mainly of glacial sands and river sand and silt deposits precluded the construction of an arch bridge. A series of comparative designs and cost estimates of various other types of bridges definitely pointed to the rigid frame type as the best solution. By the use of cantilevers and suitably balanced counterweights, it was possible not only to span the entire opening of 162 ft. but, in addition, to avoid the construction of expensive abutment and wing walls and the greater portion of costly wet excavation.

The roadway has a lateral camber of 2¼ in., measures 24 ft. between curbs and is supported on six 15 in. wide girders, stiffened laterally by struts near midspan and the quarter points. The underside of the girders is curved parabolically, to give a total depth of 42 in. at midspan. The cantilever girders have a clear span of 23½ ft., and taper parabolically from 48 in. at the end to 84 in. at the knee. In order to compensate for dead load deflection and the sag due to compression of the supporting falsework, the forms were raised above normal elevation by amounts up to ⅒ ft. at midspan and correspondingly less at other points.

The columns or legs are 24 in. deep, taper uniformly from a top width of 72 in. at the knee to 36 in. at the bottom and are laterally stiffened by a 16-in. strut wall extending from the top down to within 6 ft. of the footing. The columns bear on 18x½-in. lead plates perforated at the center to pass the inclined hinge bars. Cork slabs ½ in. thick under the outer 9 in. of bearing surface prevent bond between the column and the footing and aid in concentrating the entire load on the lead plates, thus insuring unrestricted rocking of the structure and effectively preventing the transference of moments from columns across the joint. The footings are continuous and rather wide because of the low unit bearing value of the supporting wood piles which was limited to 16 tons.

Concrete counterweights

The counterweights are suspended between, and poured monolithically with, the cantilever girders. Alternate designs proved that solid concrete counterweights cost but little more than cellular gravel pockets. Since they occupy much less volume they may be concentrated at points of greatest statical effectiveness; they obviate the construction of special floor slabs and, by stiffening the girders in a lateral direction, impart desired rigidity to the structure. The counterweights weigh about 105 tons each. Together with the supporting girders, they are so dimensioned as to cause a 50 per cent reduction in the horizontal reactions at the hinges and in the dead load moments at midspan.

The frame was analyzed for stresses due to dead and live loads, temperature variations and shrinkage, by the theory of elastic weights. This method of analysis is universal in the application to arches and frames and, with the proper simplifications it may be employed, in the case of single span structures, with a rapidity as great as is claimed for any of the more recent variants of calculation.

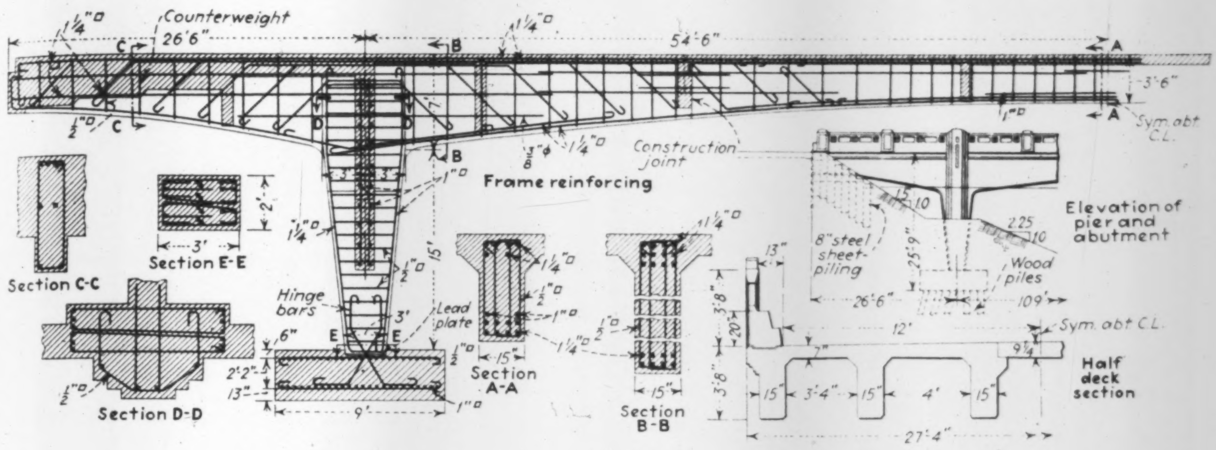
Piles for the foundation were driven in December and the pier footings and the lower pier sections were concreted under trying weather conditions in January. The subsequent exceptionally cold weather made it necessary to discontinue field operations until after March 1. This delay did not affect the design and building of the forms. Each portion of the work was studied separately to determine the best method for installing and removing the forms after the concrete had been placed and cured. The availability of an enclosed planing mill operated by the same contractor in connection with the building of the forms for the Columbus power house made practical the building of all forms for the bridge in units and in panel sections.

Concrete placing

The concrete in the superstructure was placed in six distinct sections with keyed construction joints between the adjoining units. The procedure of placing was as follows: (1) South portion of east pier and cantilever; (2) North portion of the same section; (3) South portion of west pier and cantilever; (4) North portion of the same section; (5) South portion of arch section; and (6) North portion of arch section.

During the placing of the concrete, vertical deflections were noted at various points along the span. These deflections were low and varied from ¼ to ⅜ in. maximum, depending on location.

The concrete was designed for a water content of 5¼ gal. per sack of cement and a slump of about 2 in. The aggregates consisted of sand, all of which passed the No. 4 screen, and 5 percent of which passed the No. 100 screen; and river gravel which ranged in size from 3 percent passing a No. 4 screen to that passing a 1½-in. screen. The ratio of sand to gravel was 45:55, giving an average field mix of about 1:2.5:3.1 and a cylinder strength of from 3,800 to 4,300 lb. per sq.in. at 28 days. Because of the heavy reinforcing, special care was taken to see that the concrete was


1¼" □
Counterweight
26'6"
1¼" □
54'-6"
B
A
C
C
½" □
D
D
B
3'-6"
1" □
A
Sym. abt C.L.
⅜" φ
1¼" □
Construction joint
1" □
Frame reinforcing
7'
2'
3'
Section E-E
Section C-C
1¼" □
15'
½" □
Hinge bars
3'
Lead plate
6"
E
E
2'2"
13"
9'
1" □
½" □
Section D-D
1¼" □
½" □
1" □
½" □
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15"
Section A-A
1¼" □
½" □
15"
Section B-B
13"
3'8"
20"
3'8"
7"
12'
9¼
Sym. abt C.L.
15"
3'-4"
15"
4'
15"
27'-4"
Half deck section
8" steel sheet-piling
15
10
25'-9"
2.25
10
Wood piles
26'-6"
109'
Elevation of pier and abutment

FIG. 3—STRUCTURAL DETAILS of the 109-ft. span rigid frame concrete bridge and its counterweighted cantilever side spans.

placed in its proper position. It was vibrated and spaded so well that no patching of the surface was required.

Two-span rigid frames

The two-span concrete rigid frames have 22 ft. roadways and two 5 ft. sidewalks. A longitudinal rise of ¾ in. over the center support is barely noticeable but gives a graceful line to the bridge. While the top surface is practically level, the underside is curved parabolically, with a maximum ordinate of 30 in. at midspan, giving a total slab thickness at that point of 18 in. and of 48 in. at the abutments. By Hayden's empirical formulas (*Civil Engineering*, Aug., 1935, p. 480) the corresponding dimensions would be 17 and 52 in., respectively.

The curbs are unusually heavy and were designed to meet the rigorous Nebraska state specifications for a lateral load of 1,500 lb. per lineal foot. The same specifications required that the railing be designed for a vertical load of 500 lb. and a horizontal force of 150 lb. per lineal foot. A satisfactory solution was found by combining curb and railing monolithically, thereby adding to the strength and effectiveness of the structure without impairment to the desired architectural features. Open joints of ½ in. at all intermediate rail posts divide rail and curbing into convenient longitudinal units, all tied well into the supporting slab by means of suitably located dowels.

While the inside face of the abutments is vertical, the back face is tapered from a total thickness of 42 in. at the knee to 20 in. just above the footing. Due to their greater heights center piers appear much more slender and taper from a total thickness of 36 in. at the top to 20 in. at the bottom.

The bottom joints are of the improved Mesnager (*Journal*, American Concrete Institute, March, 1935) type and are reinforced with inclined steel bars crossing at the joint plane. It is believed that this arrangement offers less resistance to rotational movement than does any other hinge, and, in keeping with the design assumptions, it effectively prevents the transference of moments from the frame of the bridge into the foundation.

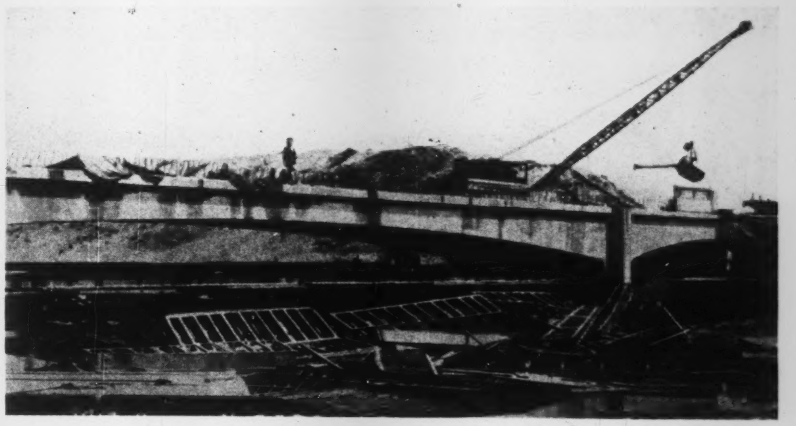


FIG. 4—WITH CENTERING REMOVED, construction begins on the composite curb and rail of the 109-ft. span rigid frame bridge.

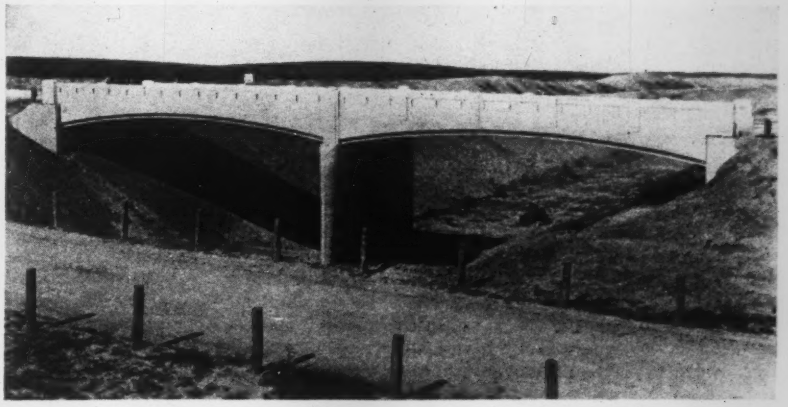


FIG. 5—ONE OF FOUR two-span barrel type rigid frames that span the canal of the Loup River Public Power District.

All footings are continuous and extend the entire width of the bridge. The supporting wood piles are spaced to carry 16 tons each, inside piles being driven vertically; all others are battered, to carry the inclined resultant loads.

The bridges are designed for a stand-

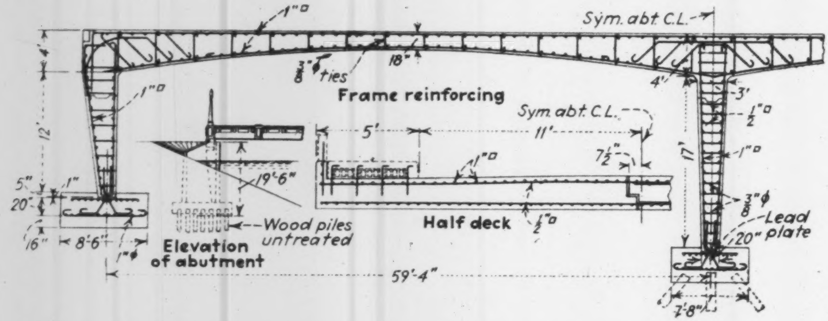


FIG. 6—FRAME AND DECK sections of two-span barrel-type rigid frames.

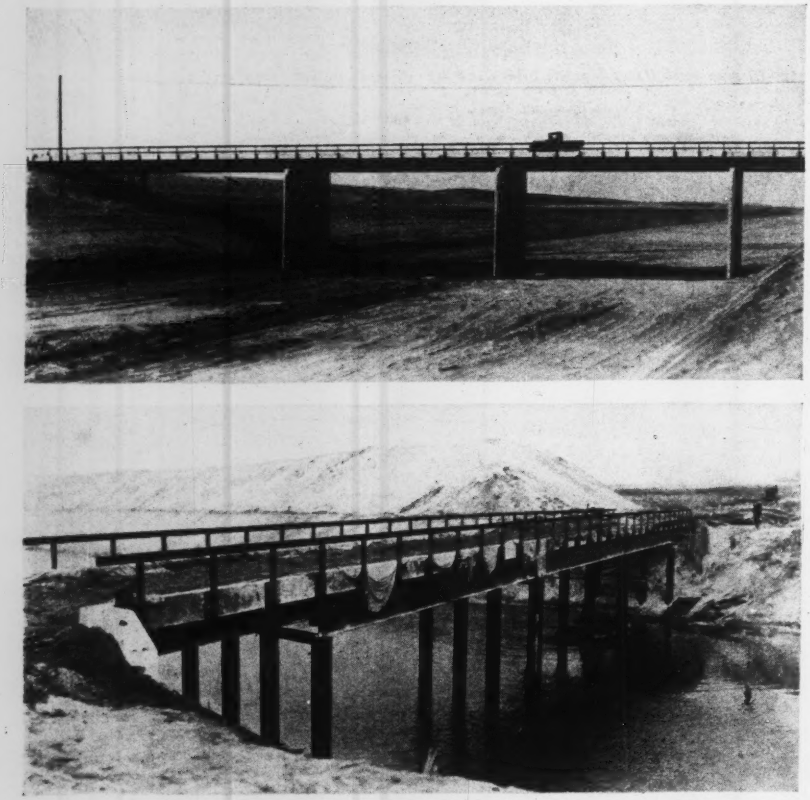


FIG. 7—TYPICAL COUNTY ROAD BRIDGES over Loup River power canal. The slender concrete piers in the upper view inclose steel H-columns and were used where groundwater levels were low enough to permit construction in the dry.

ard H-15 truck loading with proper allowance for impact in accordance with the specifications now in force in Nebraska, plus a superimposed uniform load of 15 lb. per square foot. Live load effects were evaluated by means of influence lines computed for ten equidistant points along the arch span.

Secondary effects were investigated quite thoroughly and the analysis included the determination of stresses due to

(1) Temperature variation of ± 60 deg. F.,

(2) Shrinkage, and plastic flow, which was taken as equivalent to a temperature drop of 45 deg. F.,

(3) Settlement of columns.

The secondary effects constitute a large percentage of the total stresses, being about 30 percent at the end supports, 24 percent at midspan and 34 percent at the center supports. These percentages varied for each bridge in accordance with the geometrical properties of the frame, but in each case they were sufficiently large to affect materially the amount and distribution of the reinforcing steel. Since the probability of coincidence of secondary effects is at least as great as the probability of coincident maximum static and dynamic loads it did not seem safe or logical to consider the one and disregard the other, as advocated by some writers.

In all our calculations the coefficient of thermal expansion for concrete was taken at 0.0000055, the modulus of elasticity at 270,000 tons per sq.ft. (3,750,000 lb. per sq.in.), and the vertical displacement of the column at ¼ inch.

For purposes of stress analysis the axis of the frame was taken as the locus of the center of gravity of each section and the moments of inertia were calculated for the full concrete section about this axis.

Ritter's method of fixed points ("Berechnung Statisch Unbestimmter Systeme" by A. Strassner, Berlin, 1920) was used in the final analysis of all continuous frames since it not only yields more exact results but reduces the mechanical labor of computation to a minimum. The complete analysis of a two-span rigid frame bridge for stresses due to live and dead loads and secondary effects enumerated above, including the computation, plotting and evaluation of influence lines, consumed from 25 to 30 hr. of work.

Proportioning reinforcing steel

The determination of the proper amount or spacing of reinforcing steel for combined bending and thrust in a given concrete section is usually a matter of repeated trial. A direct solution is easily possible, however, by referring all moments to the tensile or compression steel instead of to the geometrical axis of the section.

$$\text{Thus let, } k = \frac{nf_c}{nf_c + f_s} \qquad (1)$$

$$f^1_s = nf_c\left[\frac{k - m}{k}\right] \qquad (2)$$

$$M_s = N(c + e) \qquad (3)$$

$$M^1_s = N\,(c - e^1) \qquad (4)$$

$$a = \frac{M^1_s + 6f_c kd^2\left[\dfrac{k}{3} - m\right]}{f_s d\,(1 - m)} \qquad (5)$$

$$a^1 = \frac{M_s - 6f_c kjd^2}{f^1_s d\,(1 - m)} \qquad (6)$$

where, N = direct thrust; $c = M/N$; $m = d^1/d$; e = distance from gravity axis to tensile steel; e^1 = distance from gravity axis to compression steel; and all other symbols carry the conventional meaning.

Substituting in formulas (5) and (6), $f_s = 18{,}000$, $f_c = 1{,}000$, $n = 10$ and $m = 0.1$

$$a = \frac{M^1_s + 40.82d^2}{16{,}200(d)}$$

$$a^1 = \frac{M_s - 1{,}888d^2}{6{,}480(d)}$$

These formulas are easily applied, and in their application are much less liable to error than the use of the conventional tables or diagrams.

After proportioning the reinforcing steel by the above procedure and arranging it, with due consideration to the feasibility and ease of erection, unit stresses were checked by applying the well known cubic equation for $(kd)^3$. The Nebraska state specifications allow a maximum unit stress of 1,275 lb. per sq.in. for concrete and 24,000 lb. per sq.in. for reinforcing steel. The actual unit stresses were well within these upper limits and reached the maxima of 21,800 and 1,170 at the knee, and 1,245 at the crown for two-span bridges. The corresponding unit stresses for the cantilever bridge were 21,600 and 920 lb. at midspan, 20,800 and 1,250 at the knee and 19,500 and 650 just below the knee. All main reinforcing steel consisted of square deformed bars.

As might be expected, all of the rigid frame bridges are heavily reinforced. The two span bridges each contain a total of 770 cu.yd. of concrete and 64 tons of reinforcing steel, or about 166 lb. of reinforcing steel per cu.yd. of concrete. Expressed in other units, each projected square foot of bridge contains 5.2 cu.ft. concrete and 32 lb. of reinforcing steel.

The 109 ft. span bridge contains a total of 710 cu.yd. of concrete and 82 tons of reinforcing steel, or 4.23 cu.ft. of concrete and 40 lb. of steel per projected square foot of bridge. These figures include the counterweight quantities.

Concreting materials

The aggregates were obtained from pits near the confluence of the Loup and Platte rivers. Geologically they are granites, clear quartz and feldspar, with a very small percentage of limestone. They are round and smooth, but extremely hard, and both sand and gravel have a specific gravity of 2.62. Approximately a straight line gradation of the coarse aggregate, with a slight downward dip in the curve and a similar gradation in the sand gives a dense workable concrete having from $\frac{1}{2}$ to $2\frac{1}{2}$ in. slump with from 5.6 to 6.0 sacks of cement per cubic yard.

The maximum size of coarse aggregate is $1\frac{1}{2}$ in., although there is, as a rule, not more than 10 percent by weight retained on a 1-in. screen. The sand contains from three to five per cent of material finer than the 100-mesh.

The resulting concrete as placed in 6x12-in. cylinders and stored under standard conditions in moist sand for 28 days has a strength of from 3,500 to 4,500, averaging about 3,800 lbs. per square inch. The weight per cu. ft. is approximately 152 lbs., which reflects a percentage of air voids in the concrete well below 1 per cent. The concrete therefore has a characteristic of great density combined with sufficient workability to give good surface texture.

Because of the adoption of a $5\frac{1}{2}$-gal. per bag water-cement ratio, great latitude is possible in meeting the slump requirements. Internal vibration combined with the flexible slump allows the placing of concrete in the forms and around the steel in a manner so as to assure a concrete of uniform strength and strain resisting characteristics throughout the structure.

County road bridges

Twenty-two of the county bridges are of multiple-span steel construction with total lengths of from 160 to 250 ft. and roadways 16 or 20 ft. wide. They are designed for a T-15 live load, consisting of a 15-ton truck with wheels 7 ft. apart, and a distance of 11 ft. 4 in. between front and rear axles, plus a super-imposed load of 15 lbs. per square foot. The bridges have rolled I-beam superstructures with steel railings, solid concrete deck and unusually heavy concrete curbs. In order to keep the costs reasonably low, steel piling of standard H-section was used for all abutment and approach-span supports and for the center and intermediate supports of all bridges over the tailrace where the groundwater level is 10 ft. or more above canal bottom. For the center and intermediate supports of all other bridges, unusually slender concrete piers with loadsupporting enclosed steel columns were designed. The height of the piers varies from 24 to 28 ft. The thickness, however, was kept constant at 16 in. The roadway has a lateral camber of $2\frac{1}{4}$ in. and a longitudinal rise of 3 in. over the center support for 2-span bridges, and of $4\frac{1}{2}$ in. for bridges with three or more spans.

Farm bridges

In a number of cases where large tracts of land were separated by the canal from the owner's farm, the district provided access to them by erecting farm bridges of creosoted timber throughout with 16-ft. roadways and wood guardrails. All timber was detailed and billed in 20-ft. interchangeable span lengths for shop fabrication and creosoting, and shipped and erected under a separate contract. They were designed along conventional lines, primarily because experience has proven this type to be the most suitable and economical for carrying a comparatively small live load which, in this case, consists of a typical 5-ton truck.

Costs

Since the project is financed by the PWA, the following minimum wage rates per hr. prevail.

Concrete mixer	$1.20
Cement finisher	1.20
Carpenter	1.20
Form builder	.80
Painter	.90
Pile driver	1.20
Steel man	.90
Trucker	.70
Unskilled labor	.50

With these rates, and contract unit costs of $16 per cu. yd. for concrete and 25c. per sq. ft. for forms, and structural steel and steel piling furnished by the district at $63.95 and $51.00 per ton, respectively, the unit cost per projected horizontal square foot of bridge averaged $3.67 for the county bridges. For the two-span rigid frame bridges the corresponding contract prices were $12.00 per cu.yd. for concrete and $37\frac{1}{2}$c. for formwork giving a total unit cost of $5.88 per projected square foot of bridge. For the 109 lb. span bridge the contract prices were $13.50 for concrete and 55c. for framework, resulting in a total unit cost of $6.54 per projected square foot of bridge.

These costs include all expenses properly chargeable to the bridge but do not include the expenses for the non-comparable items of bridge lighting, road approaches, road and approach improvements, canal work and cofferdam.

All bridges have been designed and the erection is supervised by the Harza Engineering Co., Chicago, under the general direction of Erik Floor, vice-president and resident manager, F. E. Peacock, project chief engineer, C. E. Wattles, engineer superintendent, and the writer as designing engineer. The work is for the Loop River Public Power District, Harold Kramer, secretary and general manager, Columbus, Neb.

Provisions for Membership in Hydraulic Research Association

The provisions for membership in the International Association for Research on Hydraulic Structures as given by Herbert N. Eaton in our issue of Dec. 19, p. 855, were based on an English translation sent to Mr. Eaton from abroad. Since then Mr. Eaton has obtained the original document, which indicates that the translation sent him was inaccurate. An accurate translation follows:

Any person who is interested in hydraulic research can become an *individual member* if he is: (a) professor of hydraulics or of any related subject at an engineering college, at a university or at any other equivalent engineering institution; (b) director of a hydraulic laboratory or an employee holding an administrative position at such a laboratory; (c) member of a prominent scientific or engineering society.

Corporate membership is open to: (a) departments of hydraulics or of related subjects at an engineering college, at a university or at an equivalent educational institution; (b) hydraulic laboratories; (c) national or other committees of the World Power Conference, of the International Congress on Large Dams and of the Permanent International Association of Navigation Congresses; (d) public organizations which project, execute and supervise hydraulic works.

Determining Entrapped Air in Capillary Soils

Device for measuring air not displaced by capillary water and effects of physical and chemical composition of soils on capillary rise

By B. G. Zimmerman
Department of Chemistry, University of Texas, Austin, Tex.

Conclusion of abstract from the thesis by B. G. Zimmerman submitted to the faculty of the Graduate School of the University of Texas in partial fulfillment of the requirements for the degree of Master of Science in Chemical Engineering. The first two parts were presented in Engineering News-Record, June 13 and 20, 1935.

CAPILLARY action in soils consists in the absorption and transmission of water in all directions, regardless of gravity or other forces. The irregularly-shaped particles in a soil come in contact with each other so as to form pore spaces or voids which resemble small capillary tubes. These small tubes are not straight but are very irregular. However, the phenomenon of capillary rise of a liquid in the small capillary-like tubes in a soil is similar to that of a liquid in a glass capillary tube.

Determination of entrapped air

The volume of the capillary tubes in a soil, expressed as a fraction of the total volume that the soil occupies, is the per cent of pore spaces or voids in the soil. If the specific gravity and apparent density of a soil are known, the volume of the voids may be calculated by use of the following equation:

$$\frac{\text{Sp. Gr.} - \text{Apparent Density}}{\text{Sp. Gr.}} \times 100 = \text{Per Cent voids} \qquad (1)$$

When water enter the capillary tubes, most of the air is displaced because soil is wetted by water, and hence the latter is drawn into all capillaries open at the farther end to allow the air to be pushed out. However, not all the capillary tubes in a soil are continuous; some of these are in the form of small sacks. These small sacks cannot be filled by the capillary water because the entrapped air will be compressed until it develops sufficient pressure to counteract the force with which the water is drawn into the capillaries.

To determine the volume of air that is trapped in these small sacks by the water as it enters their mouths, the apparatus shown by Fig. 1 is employed. This apparatus may be called a voidometer. The specific gravity of the soil is determined before it is compacted into the soil tube. Each tube is sealed at the bottom with a filter paper, and the compaction is accomplished by tapping the tube as the soil is being poured into it. The weight of the soil is obtained by determining the difference in the weight of the tube before and after filling it with soil. Since the tube which is being employed in the experiment is of Pyrex glass and of fairly uniform bore, the volume of the soil can be determined by calculating the capacity of the soil tube from its dimensions. The apparent density of the soil is then calculated by dividing the mass by the volume.

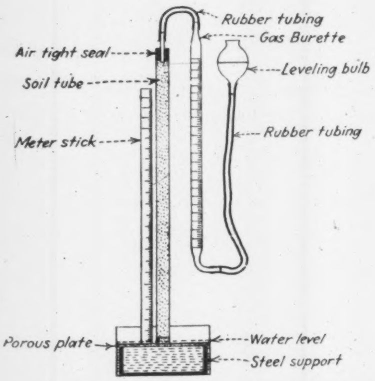


FIG. 1—VOIDOMETER for determining volume of air not displaced by capillary water in soils.

The top of the tube is connected to a gas burette, which is filled with a fuel oil saturated with air. The soil is permitted to take up water by capillary action and the air which is displaced by the rising water is collected over the fuel oil in the gas burette. As the water in the soil tube rises through any particular height, the volume of air displaced is determined by means of the gas burette. This volume of air corrected to the pressure and temperature at which the specific gravity of the soil was determined, is the actual volume of the water in this particular height or amount of soil. A number of observations are made as the level of the water rises in the soil tube.

The volume of the water absorbed by the soil may be calculated from the following equation:

$$V_1/V_2 \times 100 = X \qquad (2)$$

where V_1 is the corrected volume of the air displaced by the water, V_2 is the volume of the soil wet with the capillary water, and X is the percentage of the volume of water in the soil.

Since the percentage of the voids in the soil may be calculated from equation (1), only the percentage of voids not filled with water remains to be calculated. This may be calculated from the following equation:

$$V_p - X = X_1$$

where V_p is the percentage of the voids in the soil obtained from equation (1), X the percentage of volume of the water in the given amount of soil obtained from equation (2), and X_1 the percentage of voids not filled with capillary water. If the voids are completely filled with water, then the value of X_1 will be equal to zero.

Tables I and II contain the results of two experiments made to determine the percentage of voids not filled with capillary water. Separation of the different sized particles of Sample No. 1 is accomplished by screening the material and taking two fractions collected between two consecutive screens. Fraction A represents the material passing a No. 30 screen and retained on a No. 40 screen, whereas Fraction B represents the material passing a No. 100 screen and retained on a No. 200 screen. The average uniform diameter of Fractions A and B are 0.50 and 0.11 mm. respectively. Fraction A is used in obtaining the data in Table I and Fraction B that in Table II.

The results obtained from these experiments indicate that not all the pore spaces in a soil are filled with capillary water. This is probably due to the noncontinuous tubes which form small sacks from which water can not displace the air. Furthermore, an increase in particle size generally indicates a decrease in the percentage of voids not filled with water, and finally the percentage of voids not filled with water is increased with an increase in the height of rise of the water in the soil. This phenomenon is probably due to the fact that the water is approaching the equilibrium which is established between the weight of the water above the general water level and the upward pull of the water along the walls of the capillary tubes.

Rate of capillary rise

It is generally believed that the rate of capillary rise increases with an increase in particle size but the ultimate total rise is decreased. In other words, the rate of capillary rise is more rapid for coarse than for fine soil textures.

Collins and Stewart (*E.N.-R.*, 112, No. 21, 661, 1934) in a recent article

TABLE I—ENTRAPPED AIR FOUND IN FRACTION A OF SAND GRAINS HAVING A DIAMETER OF 0.50 MM. USING A DIAMETER OF TUBE OF 1.524 CMS.

Sp. Gr. = 2.62 Per cent voids = 39.70 Density = 1.58

Temp. degrees C.	Barometer reading in mm.	Rise in cms.	Gas burette reading	Per cent voids not filled withwater
26	744.0	0	97.0	....
26	744.0	10.1	90.2	2.80
26	744.0	11.0	89.8	3.80
26	744.5	13.8	88.2	4.70

TABLE II—ENTRAPPED AIR FOUND IN FRACTION B OF SAND GRAINS HAVING A DIAMETER OF 0.11 MM. USING A DIAMETER OF TUBE OF 1.524 CMS.

Sp. Gr. = 2.63 Per cent voids = 39.20 Density = 1.60

Temp. degrees C	Barometer reading in mm.	Rise in cms.	Gas Burette reading	Per cent voids not filled with water
26	744	0	98.0	....
26	744	40.3	72.7	4.80
26	744	47.5	69.2	6.00
26	744	59.8	64.5	8.50

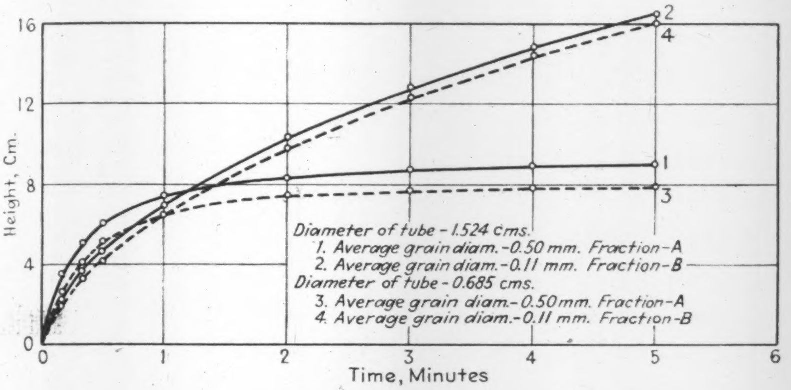
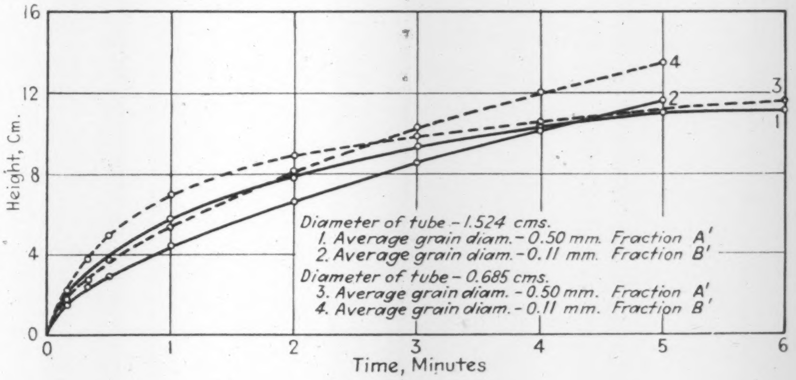


FIG. 2—CURVES showing effect of soil particle size on capillary rise of water.

FIG. 3—CURVES showing effect of chemical composition of soils on capillary rise of water.



on the theory of soil stabilization do not seem to agree with these generally accepted observations. They state that silt particles which are smaller than fine sand transmit water more rapidly. While the writer's (*E.N.-R., 114,* No. 25, 876, 1935) experiments determined that silt particles are smaller than fine sand, he sees no reason why fine silt should show a behavior countrary to the results presented in this paper. From Figs. 2 and 3, where two different types of soils are employed, it can be seen that in each case the coarser particles transmit water more rapidly. Several additional experiments were carried out in this laboratory with different types of soils and in each case it was found that soils with coarser textures transmitted water more rapidly than those with finer textures.

From Fig. 2 it can be seen that the ultimate total capillary rise is greater in the large diameter tube (Curves 1 and 2) than in the small diameter tube (Curves 3 and 4). Wadsworth and Smith (*Soil Science, 22,* 199, 1926) in their experiments on the extent of capillary rise of water through fine sandy loam soil observed that after 263 days it was about 65 per cent greater in columns which had a cross section of 25 sq.in. than in columns which had a cross section of 1 sq.in. No satisfactory explanation can be supplied for the above results.

Since it has been found by Bigelow and Hunter (*J. Phys. Chem., 15,* 367, 1911) that the capillary rise of a given liquid is dependent upon the walls of the capillary tube, it is to be expected that the chemical composition of soils affect the extent and probably also the rate of capillary rise in soils.

In the following experiment two soils which have different chemical composition are employed. One type of soil employed is represented by two different sized fractions A and B of a sand and the curves are presented in Fig. 2. This sample is designated as No. 1 and contains 99.10 per cent silicon dioxide. The other type of soil is represented by two different sized fractions A′ and B′ of a clay and the curves are presented in Fig. 3. This sample is designated as No. 2 and contains the following constituents:

SiO_2	63.36%
Al_2O_3	24.42%
Fe_2O_3	1.45%
CaO	0.37%
MgO	0.47%
Loss on ignition	9.95%
Organic matter	0.00%
Total	100.02%

Since the grain sizes of Fraction A are equal to A′ and those of B equal to B′, their corresponding curves should agree within experimental error provided that capillary rise is dependent only upon particle size. However, this is not the case and by comparison, it can be seen from Figs. 2 and 3 that the rate of capillary rise in fraction A of the sand (Fig. 2) is greater than that of Fraction A′ (Fig. 3). Since capillary rise is undoubtedly dependent upon the walls of the tube, it may be concluded that a difference in soil composition will also exhibit a difference in capillary rise.

Many helpful suggestions were obtained from Dr. E. P. Schoch, Professor of Chemical Engineering at the University of Texas.

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All-Welded Building in Turin, Italy

AN ALL-WELDED apartment building involving 795 tons of steel has recently been completed in Turin, Italy. Known as La Tour Littoria, the structure consists of a 9-story base about 150 ft. long and 75 ft. wide surmounted by a 10-story tower 50x43 ft. in plan. The roof of the building is 240 ft. above the street, and there is a campanile 40 ft. high on top. Rolled section beams are used while the columns consist of two I-sections joined together by narrow plates placed on 3 ft. centers.

FORT PECK DAM TODAY—II

Dredge Plant Characteristics And Hydraulic Fill Operations

H. W. Richardson
Associate Editor, Engineering News-Record

THE FORT PECK DAM dredging plant has been proved. Any doubts that might have existed as to the ability of the dredges effectively and efficiently to pump material to the top of the dam, 230 ft. above the river, has been dispelled by the remarkable success this season in pumping fill for the dike section that extends along the beach for 11,500 ft. westward from the main dam. One of the four 28-in. hydraulic dredges, the *Jefferson,* has been assigned to pumping into the dike since the opening of the current dredging season on Apr. 19. In two months a total of 1,947,000 cu.yd. of material had been pumped into the dike against a head of over 200 ft. Most encouraging is the fact that the operating characteristics of the *Jefferson* working against the high head compare favorably with those of the other three dredges pumping into the dam against a much lower head. For example, a daily dredge report picked at random reveals that the *Jefferson* pumped 1,921 cu.yd. per hr. while all four dredges averaged 1,811 cu.yd.; her pipeline velocity was 24.7 ft. per sec., the average was 22.4 ft.; she consumed 5.08 kwh. power per cu.yd. pumped, the average for the four dredges was 5.38 kwh.

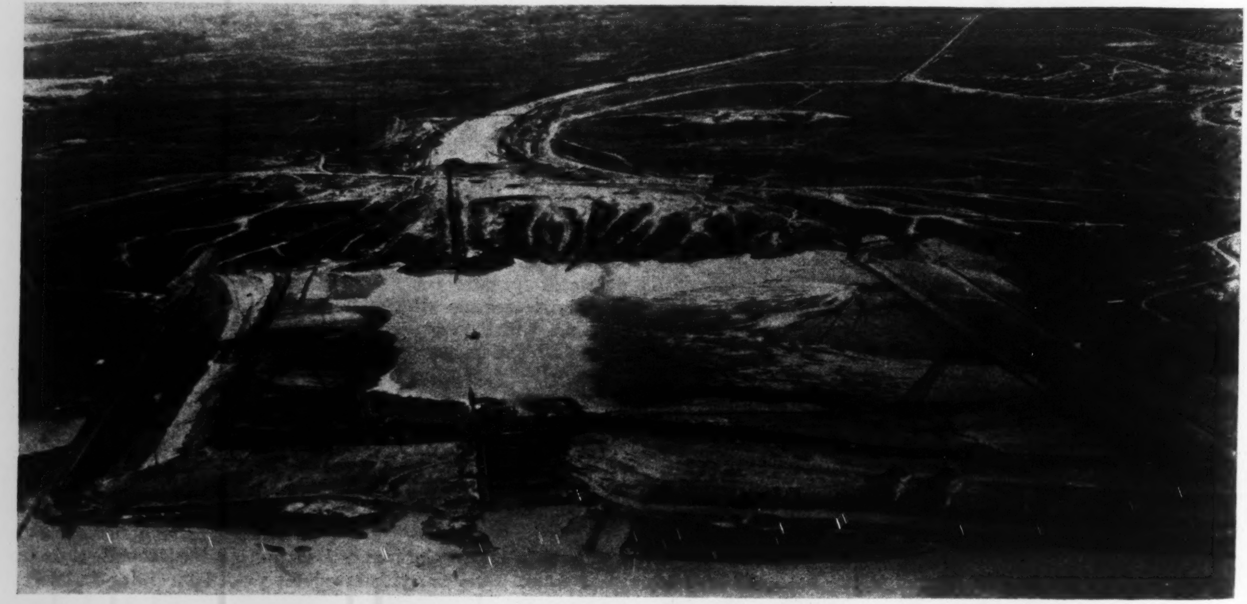
So far, this season's dredging work is an improvement even over last year's remarkable showing, when 25,259,000 cu.yd. of material were pumped in the 6½-month dredging season. Of this quantity, 20,899,000 cu.yd. were retained in the dam. In May of this year, a record placement of 3,734,000 cu.yd. was made, which required the pumping of 4,467,000 cu.yd., the difference being lost over the corepool wasteways and otherwise. Last year the highest month's yardage was 3,600,000.

Description of plant

The Fort Peck dredging plant consists of four similar electric-powered hydraulic dredges, named the *Gallatin, Jefferson, Madison* and *Missouri.* Each dredge is equipped with two 28-in. centrifugal pumps set in tandem and individually driven by 2,500-hp. wound-rotor induction motors. At the end of a 75-ft. ladder is a 7-ft. dia. basket-type cutting head, designed for digging soft materials, driven by a 500-h.p. motor at a constant speed of 26 r.p.m. Auxiliary equipment includes service water pumps, main motor speed control devices, five-drum winch for swinging and hoisting the ladder and two cast steel spuds 75 ft. long, and a power-cable reel. Operation of the swing cables, spuds and ladder is from one electric motor. A more complete description of the dredging plant design was given in *ENR* Dec. 12, 1935, p. 810.

Connected into the pipeline from each dredge is a floating booster plant, containing two 28-in. pumps in tandem, and otherwise similar to a dredge, except, of course, the cutter head and spuds are lacking. The boosters are usually located in dredged slips, roughly half way between dredge and dam. Next to the dam is a land booster for each dredge, a rail-mounted pumping unit containing but one 28-in. pump with the necessary

FIG. 1—HYDRAULIC filling operations at Fort Peck in full swing. View of left bank section of the main dam with dike section in background. The Missouri River, dividing the dam into two sections is in the foreground. Note the stepped wooden wasteways for the core pool water and the discharge pipelines carried on trestles.



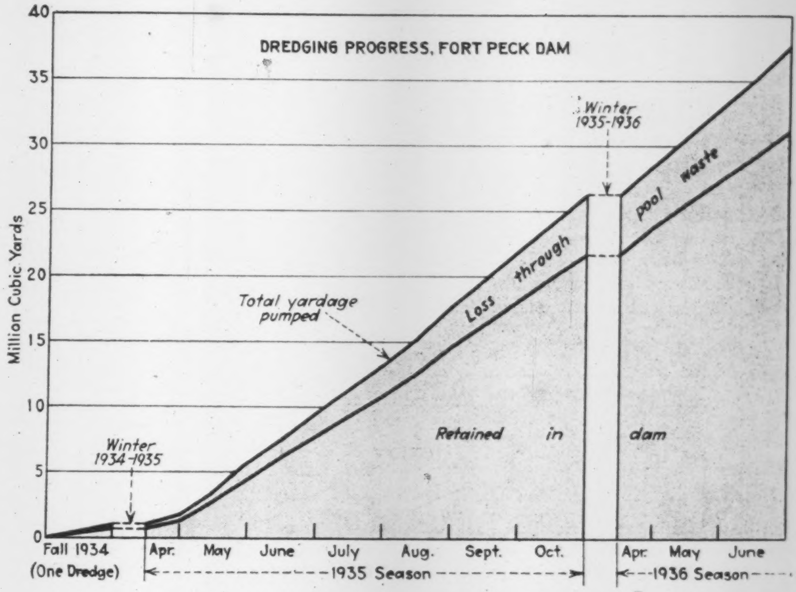


FIG. 2—OVER 30 MILLION cu.yd. of material has been placed in the dam to date, requiring the pumping of 37,500,000 yd.

auxiliary equipment Thus, each dredging unit contains five interchangeable pumps. One complete pump is kept as a spare for every pump in service, making a total of 40 pumps on the job.

Two types of pipeline are in service, both 28-in. diameter. From dredges and floating booster to land, the pipe (carried on pontoons) is flanged, ¾-in. wall thickness, with flexible ball joints. Special landing barges carry the transition between floating and land lines. On land, the lines are long straight sections of bolted flanged pipe. From the start, great care has been taken to set the land lines to exact grade and alignment, using blocking where necessary to maintain an even grade. Experience in reduced wear of pipeline and lower friction losses has proved the economy of such precautionary measures. Discharge lines on the fill are quick-connecting ball-and-cone sections, 12½ ft. long. Joints are held by toggle connections, and the ball-and-cone feature allows considerable flexibility in the line.

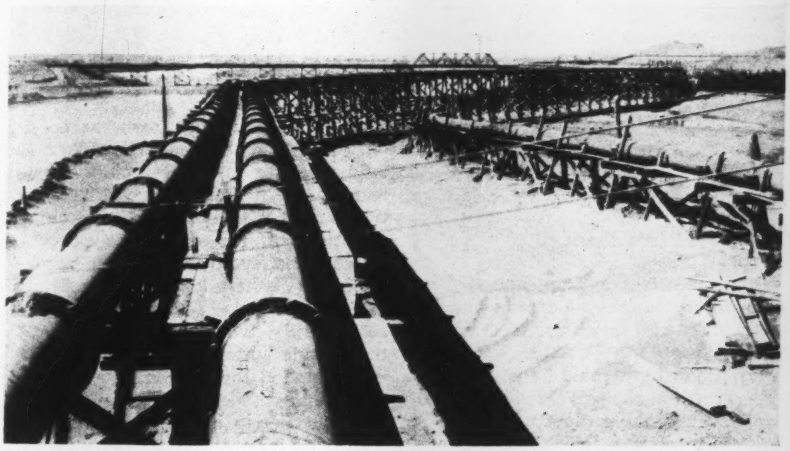


FIG. 3—STRAIGHT PIPELINES carried to even line and grade on trestles transport pumped material across the broad expanse of the right bank section of the dam. At the right is a special line for building up the sand plug at the river end of the silt core.

The servicing of the dredging plant requires a good-sized fleet of floating derricks, tow boats, work barges, launches and pontoons. Except for one towboat, which took a year to come up the river from Kansas City, and nine launches, all of the floating plant, including the dredges, was built or assembled on the job at Montana's lone boatyard. Most of the plant was designed by the army engineers, and the various parts were manufactured to order.

Increasing performance

At the start of the 1935 season the dredging plant was new and untried, except for a brief period of operation of one dredge late in the fall of 1934. The dredges were different in several respects to any previously built. Project labor regulations permitted bringing in key men and certain operators and mechanics only, the rest of the labor was recruited from within the state of Montana.

With a stiff, new plant, manned partly by operators experienced in their line but unfamiliar with the equipment at hand, and partly by raw recruits from the mines, farms and ranges of Montana, the army engineers started the unprecedented job of building a 100,000,000-yd. fill in five short seasons. The dredging plant was designed to meet a schedule of 3,000,000 cu.yd. per month. By midsummer last year the plant was working smoothly and was slowly gaining on the schedule.

Throughout last year the performance of the dredges steadily increased, exceeding all expectations. At the start of the current season the rise in efficiency and production was carried on from where it left off last fall. A study of the operating characteristics of the plant, plotted in Fig. 4, will show the continued increase in performance.

The most significant curve is that showing the average yardage pumped per hr., which steadily rose to a peak of 1,785 cu.yd. per hr. per dredge during the final two-week period of 1935, and started out this year with 1,818 cu.yd. per hr. Due to pipeline changes, shifts of dredges and changes in equipment, the output dropped to 1,710 cu.yd. per hr. early in May, but is slowly rising, the first June period showing 1,743 cu.yd. The average last season for all dredges was 1,628 cu.yd. per hr.; so far this year it has been 1,747 cu.yd.

The per cent of solids pumped has been maintained at a fairly steady average. Last year the average was 13.4 per cent; this year it is 13.2 per cent. Character of materials in the borrow pits is the greatest factor in this item of performance; sands and silts make for higher percentage than do clays and gravels.

Power consumption per cu.yd. of material pumped naturally rises with increasing lengths of pipeline and with increasing pumping heads, though certain changes in dredge machinery this year have reduced the power load to some extent Last year, the consumption of power varied from 2.99 to 4.90 kw.-h. per cu.yd., the average being 3.85 kw.-h. This year, because of the higher lift on all dredges, especially that of the *Jefferson,* the power consumption has averaged 5.55 kw.-h. per cu.yd.

Last year, the vacuum on the forward

OPERATING CHARACTERISTICS, FORT PECK DREDGING
(Unit Average of Four Dredges, Semi-monthly Periods)

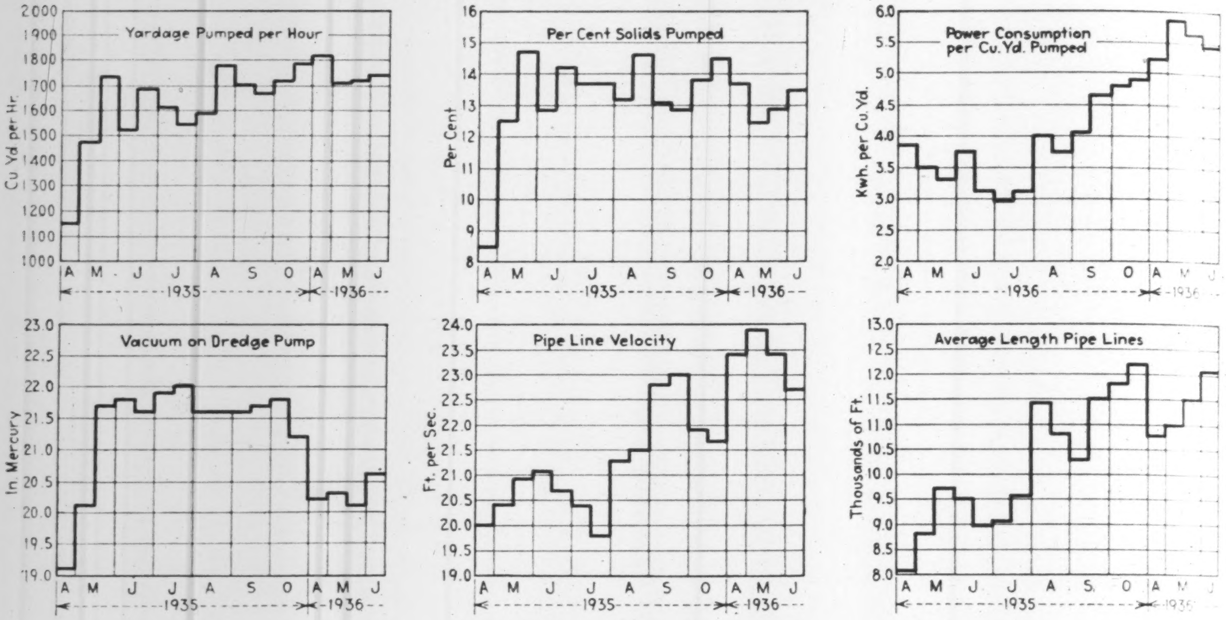


FIG. 4—CHARACTERISTICS of dredging operations plotted for two-week periods from the beginning of last year's season to date.

pump of each dredge was maintained as high as possible, just below the breaking point. During one two-week period the vacuum averaged 21.9 in. of mercury, though the year's average was 21.4 in. This year, an attempt is being made to obtain the same or greater yardage with a lower vacuum, certain changes in equipment making this possible. To date the average this season has been 20.3 in. With the lowering of the vacuum the velocity in the pipelines has been increased. Last year the velocity ranged from 20 to 23 ft. per sec.; this year it has been stepped up to an average of 23.4 ft. per sec.

The last curve in the chart shows the average length of pipeline for each dredge. At the close of last season the average length was 12,200 ft., while the average for the year was 10,100 ft. As rapidly as possible the lines were stretched out again this year, at present averaging 12,200 ft. again, and the dredges are working in the far reaches of the borrow pit. As the season advances the dredges will work towards the dam, and the resulting decrease in pipe length in a measure will compensate for the increasing pumping head. A daily report of dredging operations is shown in the accompanying table.

Plant changes

While the dredging plant operated in a satisfactory manner last year and by mid-season was exceeding its designed capacity, those in charge felt that greater production would be possible by making certain changes in design and arrangement of the equipment.

Last year, it was found that when a lens of clay was encountered by the cutter head a momentary plugging of the suction line intake resulted, causing the vacuum to break with undesirable surges in the pipeline. In fact, one such surge broke the line between pumps on a floating booster, and before the dredge could be shut down the booster barge filled with mud and sank. The only recourse in case of plugging was to slow down the pumps until the suction cleared. This required the constant attention of the dredge operator to watch the vacuum gage and to regulate the speed of the pumps accordingly.

A simple but effective measure corrected this trouble. Last August a three-blade butterfly valve, air-operated, was installed low in each suction pipe, close to the intake. Control of the valve is by pedal trip in the operating house. The operator watches the vacuum gage at all times, and when the vacuum nears the breaking point, indicating incipient plugging of the intake, he trips the valve allowing clear water to be sucked into the line. Thus the vacuum is held until the suction end clears. Disruptive surges have been eliminated, and the pumps now operate at practically constant speed. An opening of 6x12 in. in the butterfly valve is most effective.

FIG. 5—TWO UNITS of the dredging plant. At the left is the dredge *Missouri*, one of four that combined pump an average of 160,000 cu.yd. daily. Pump changes are now made in 8 hr., instead of 15 as required last year, for facilities now permit handling the units intact, as the view on the right shows.

While surges originating at the dredge have been overcome by the trip valve installation, occasionally minor ones occur in the lines because of disturbances at the booster pumps. Differences in pump speeds and other factors sometimes create undesirable vacuum and pressure fluctuations. A home-made relief valve has ended this trouble, not by eliminating the cause, but by protecting the pipelines when the line surges occur. On the suction side of the pipelines close to the boosters, the pipelines have been tapped for a 6-in. nipple, capped by a quick-acting valve. The free side of the valve is blanked by a thin metal plate bolted to the flange.

The valve is normally left in open position, and in case of a pressure surge exceeding 300 lb. per sq.in., the metal diaphragm ruptures, instantly relieving the pressure within the line. When this happens, the booster operator closes the valve and replaces the broken diaphragm with a new one in a few moments.

Two other changes in the dredge plant are of interest, which are in no way a reflection upon the original design, for when the dredges were designed there were no experience data available on units of this size or height of lift involved. When the dredges were built both pumps were placed at deck level of the barge. Recently the forward pump on the *Missouri* was dropped to the bottom of the barge, a drop of 37 in., thereby reducing the height of suction. The change was relatively simple, requiring only local strengthening on the barge bottom, and rotating the pump a few degrees to accommodate special connections required. This experiment has proved successful in increasing efficiency and production. Similar changes are being made on the other three dredges as rapidly as possible.

Experiments in suction-pipe sizes have shown that a larger pipe than that originally installed is more effective. At first the suction was 30 in., but early last year all suctions were changed to 33-in. pipes. This year, again using the *Missouri* for testing, the suction size was successively increased to 35 in. and then to 38 in. The 35-in. size has been adopted, and all dredges will be fitted with this size suction in the near future.

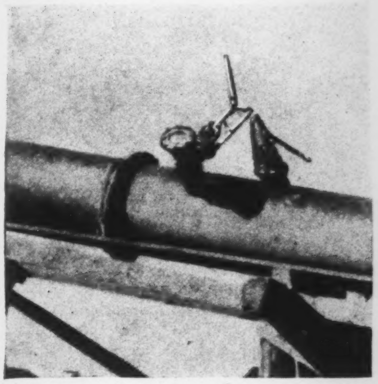
The use of flanged pipeline laid to even line and grade has been found so effective in reducing friction losses and pipe wear that the use of this type of installation has been extended to the utmost. Today, the lines running across the dam and along the river channel to the downstream beaches, and the feeder extensions along the berms to the abutments are all of this type, carried on semi-permanent trestles. Building of these trestle lines and the overhaul of dredging and other equipment provided winter work for a large crew which otherwise would have been idle during the shut-down of pumping.

Through the use of alloy metal liners and by building up worn pump parts with hard-surfaced welding, the service life of a pump has been lengthened from three to four weeks before repairs are required. With a view to further increasing the run between pump changes, bids are being called for on metal-backed rubber rings to fit into points of greatest wear. The use of rubber is admittedly an experiment never before tried on pumps of this size, but one pump is being altered to receive the rubber rings for experimental purposes.

Pumps changed quickly

Another increase in operating efficiency of the dredges is the speed in which pump changes are made. Formerly, this took more than 15 hr., but handling facilities have been improved to permit removal and replacement of pumps as a whole, cutting the changing time down to 8 hr. Pumps are repaired and rebuilt at the job in one of the best-equipped welding shops in the country (*ENR*, Apr. 30, 1936, p. 624).

FIG. 6—DISRUPTIVE SURGES in the pipelines have been tamed by these simple but effective relief valves of light metal flange disks that burst when the pressure exceeds 300 lb. per sq.in.



All of these changes have helped toward better performance, increased production and higher efficiency of the dredging plant. When contemplated changes have been made on all of the dredges, the army engineers are looking forward to an output of 4,000,000 cu.yd. per month, not an impossibility in view of the yardage placed during May.

However, the greatest factor in dredge production is the character of materials encountered in the borrow pit. Thousands of borings were made in the pits, and from their logs horizontal and vertical sections were plotted to show the character, location and extent of the various classes of materials (*ENR*, Aug. 29, 1935, p. 285). Every endeavor is made to keep the dredges cutting into desirable material. Upon the success of this endeavor rests the real efficiency and progress of the operations.

Current year's operations

As was the case last year, all four dredges are working in upstream borrow pits and will continue in this location until just prior to closing of the center section of the dam next summer, when they will be moved downstream from the fill. The *Jefferson* is pumping into the dike at present, but will be finished there early in August. The *Gallatin* is pumping into the west bank section, while the *Missouri* and *Madison* are working on the much larger east bank section.

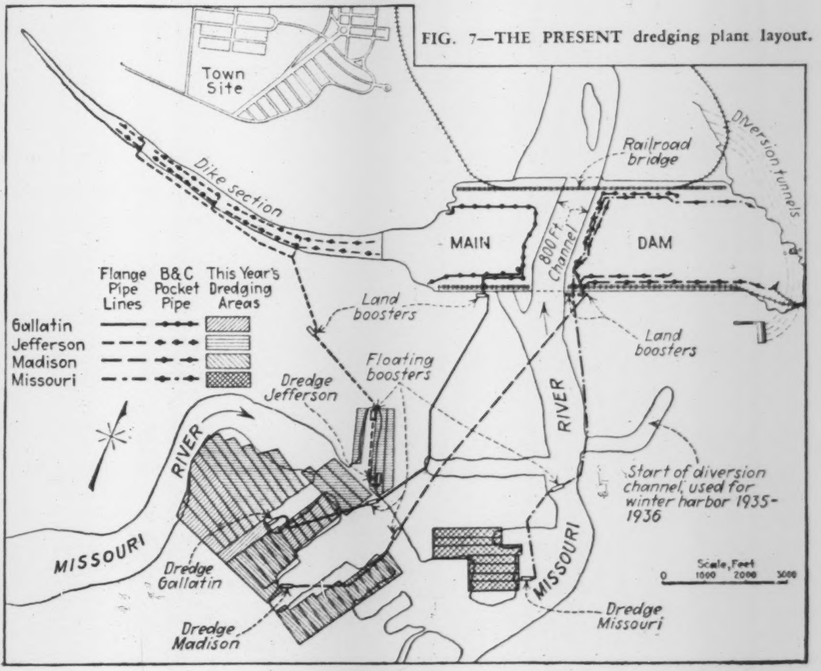


FIG. 7—THE PRESENT dredging plant layout.

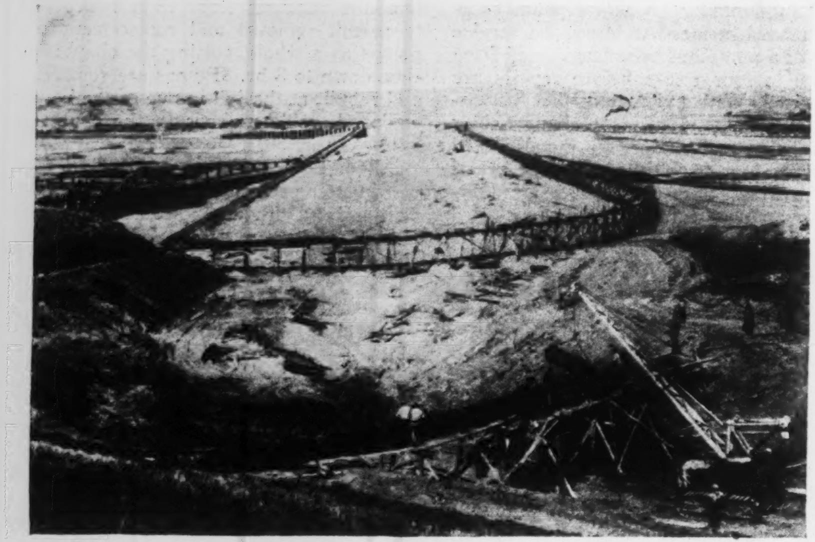


FIG. 8—COULEE POCKETS are filled from the upper end. This crew is setting up a pipeline for discharging at the extreme western end of the dam. All discharge lines are now carried on trestles. The curving line in the distance is the sheetpile cutoff wall on the east abutment.

For the most part, dredging and filling operations are the same as last year (*ENR*, Aug. 29, 1935, p. 292). The heavy flanged-type discharge lines are carried up on the fill as far as possible, a consideration being that they should be so placed that frequent moves are not required. From there the lighter ball and cone pipelines are carried along the beaches. Lifts average 6 ft., temporary trestles being built as each lift is put into place. When dredging commenced, retaining dikes were essential for each lift inasmuch as the fill could not be placed directly against the gravel toes and also since other operations, such as cut-off wall construction, restricted the dredging areas. As the fill was built up and the entire base of the dam could receive fill, the dikes were abandoned in favor of the use of shutter, or as it is sometimes called, pocket pipe.

Each pipe section has been fitted with a 6x12-in. trap door in the bottom, closed by a sliding gate. During discharge operations about 35 consecutive traps are left open, through which 80 per cent of the material escapes, the remainder going on through to the end of the pipe, discharging upon tables. As the fill builds up under the pipe, the line of open traps is moved, one pipe length at a time by closing a trap at one end of the discharge section and opening up a new one at the other end. The trestles are built in 6-ft. lifts, each lift being moved inward from the face a distance to correspond with the side-slopes of the dam. A power auger mounted on a tractor bores shallow holes in the fill to receive the trestle posts, which are placed by a tractor crane. Pipe sections are transferred from old to new trestles by the same crane. When a trestle has served its purpose in one location, all timber is removed an re-used.

Naturally, the material under the di charge line builds up in a low ridg throwing part of the discharge towar the outside By careful manipulation shear boards the coarse material fallin to the outside of the ridge is used t build up a natural levee, and the use draglines for this purpose is no long required (Fig. 10).

The ends of the dam are now wor ing up into the coulees and breaks the abutments. These pockets are fill from their upper end, as shown Fig. 8.

The sand plugs built up to restra the ends of the core next to the river the two sections of the main dam hav been flattened to a slope of 5:1 to pr vide greater stability. In these plu are the core pool wasteways, tw wooden sluices of 5-ft. staggered step carried on wood piling and protected wakefield sheetpile cutoff walls. Mat rial for the sand plugs is placed fro

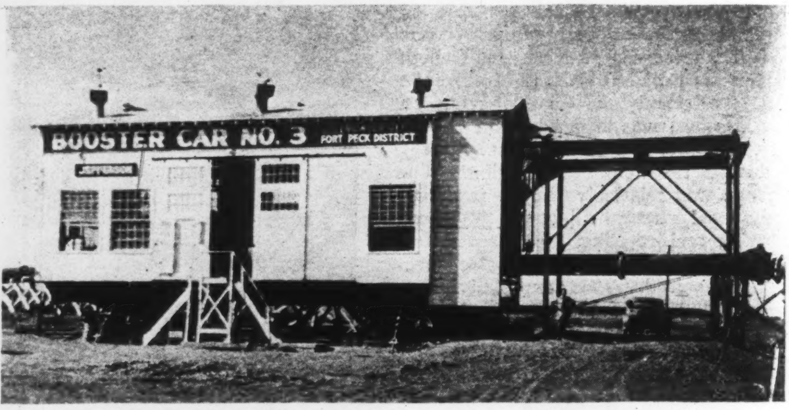
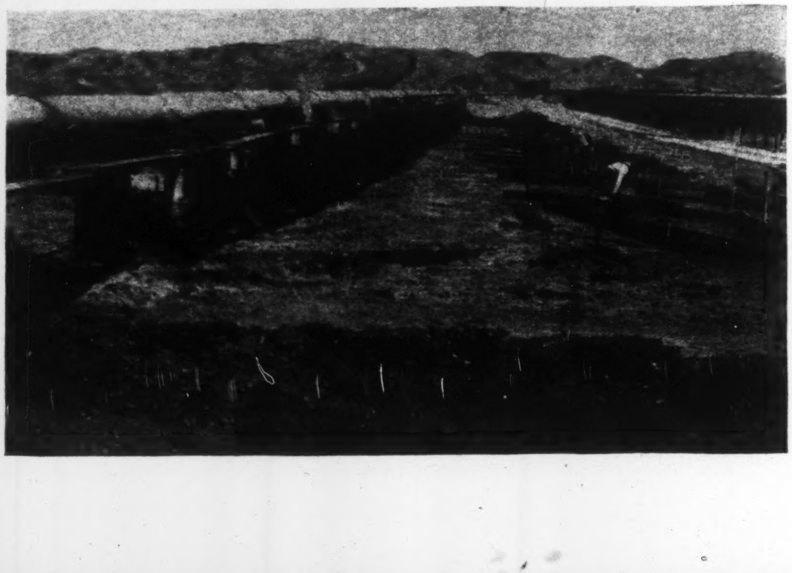


FIG. 9—THE BOOSTER that climbed a hill. This is the land booster unit used in the dike pipeline. The gantry frame at the right is a convenient aid in removing and replacing the 28-in. pump.

FIG. 10—TRAPPING through pockets in the bottom of the trestle-mounted pipelines is the current method of discharging materials into the dam. The shear boards at the right build up the outside levee, formerly done by draglines. At the extreme right is a flanged feeder line leading to the east end of the dam.



bypass pipelines taken off from the 28-in. main (Fig. 3).

Building the dike

Because of the phenomenal progress being made, about 43,000 cu.yd. per day, the most interesting filling operation today is the construction of the 11,500-ft. dike at the west end of the main dam. Of the 3,240,000 cu.yd. required for the dike, about 3,000,000 yd. is hydraulic fill, extending for nearly two miles. The remainder, all above the maximum flow line of the reservoir, will be truck fill rolled in place.

Two factors influenced the decision to pump the dike, originally intended to be built entirely of rolled fill. Progress on the dam is beyond all expectation, and as a result the embankment is rising faster than at first contemplated. Of course this increases the height of the sand plugs restraining the core. By throwing one dredge to dike pumping, progress on the main dam could be safely slowed down somewhat without sacrificing the progress on the whole or the efficiency of operations. At present rate of pumping into the dam, the sand plugs will be as high as desired from a safety standpoint by the time the river is diverted. The other factor in favoring the pumping of the dike is that such operations will test the dredging plant thoroughly against high heads, and any faults can be corrected before the main dam rises to similar heights. The cost of hydraulic filling has been found to be less than that estimated for dry fill.

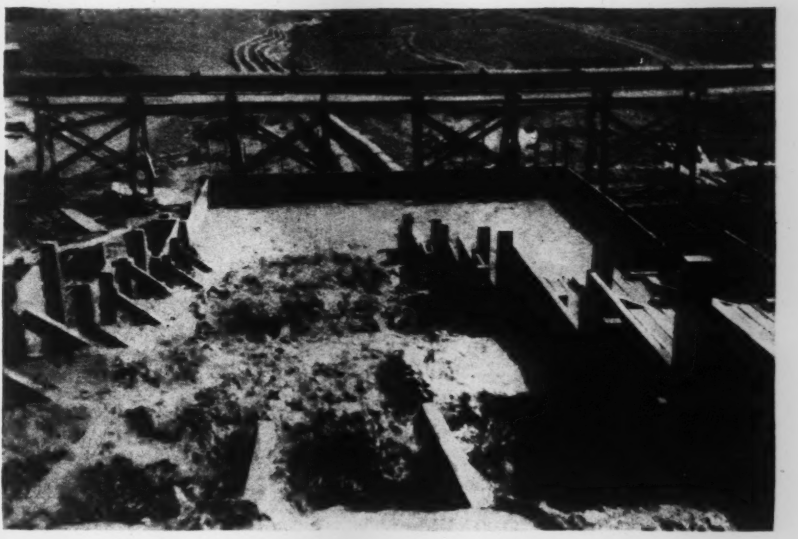


FIG. 13—A NEW TYPE of core pool spillway is used at the dike, with a sump box at the lower end of stepped sluiceways. Two 28-in. pipes drain the sump, passing under the main feeder line for the dike.

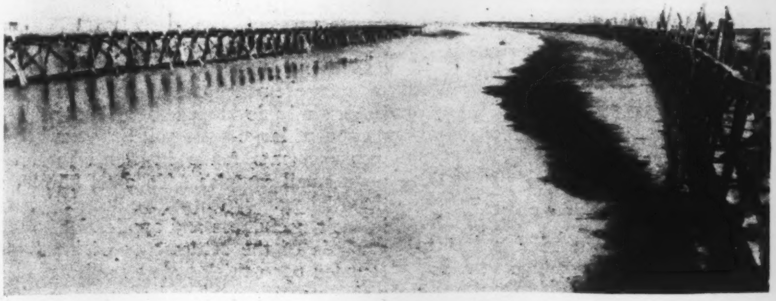
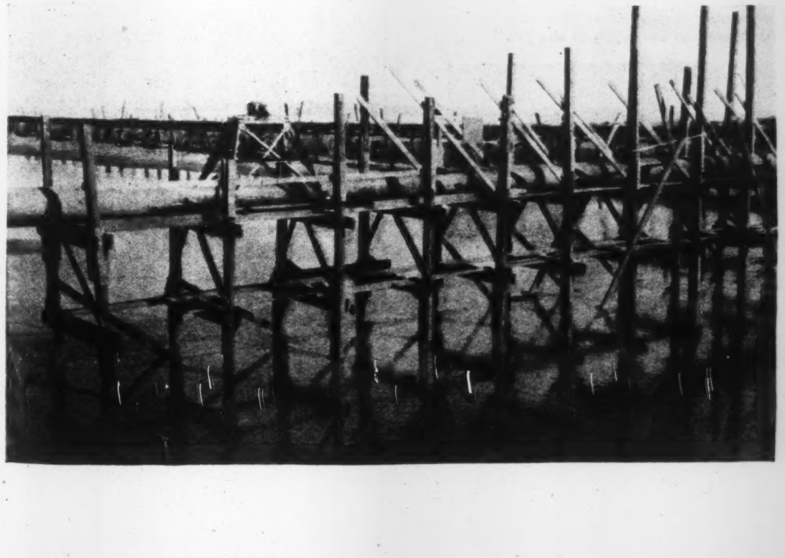


FIG. 11—HYDRAULIC filling of the dike is progressing at the rate of 43,000 cu.yd. per day. This placid stretch of water is the dike core pool, lined with pocket trap discharge pipes.

FIG. 12—IN THE LOWER VIEW is one of the cross lines tapping the discharge pipe on this far side of the fill from the feeder line. The small gantry hoist, rail-mounted, hustles the pipe sections to shore when the trestle is raised.



The dike varies from 55 to 5 ft. in height, has a top width of 50 ft., and sideslopes of 4:1 to reservoir line, 3:1 for the 20 ft. of freeboard. The silt core is carried to reservoir elevation with a 15-ft. top width, sideslopes are $\frac{2}{3}$:1 and $\frac{1}{2}$:1.

For the dike work the *Jefferson's* land booster was hauled up the steep slope of the west abutment to the bench upon which the dike rests (Fig. 9). A small reservoir made in a nearby coulee, supplies service and cooling water for the land booster.

The first step in dike construction was to build a gravel toewall 5 ft. high on the downstream side for drainage of the fill This was done last winter by truck haul, the trucks dumping from a portable timber ramp that permitted direct placement of the gravel. Then the inside face of the toe fill was lined with sandbags to confine the core pool water until the core was started.

A flanged pipeline was built on a trestle outside of the fill on the upstream side for full length of hydraulic operations. From this main pipeline, service lines on temporary trestles lead to both sides of the fill at 1,000-ft. intervals, connecting with pocket pipe fill lines similar to those used on the main dam.

DAILY REPORT OF DREDGING OPERATIONS

For Period Ending 8 A.M., June 19, 1936

Dredge	Gallatin	Jefferson	Madison	Missouri	Total or Ave
Advance—ft.	67	118	94	85	364
Dredged—cu.yd.	47,100	43,840	38,310	33,630	162,880
Pumping time	21h. 19m.	22h. 49m.	23h. 35m.	22h. 14m.	89h. 57m.
Cu.yd. per hr.	2,210	1,921	1,624	1,513	1,811
Ave. pipeline vel.—ft. per sec.	21.8	24.7	22.3	20.9	22.4
Per cent solids	17.8	13.6	12.8	12.7	14.2
Power—kwh.	202,200	222,600	208,900	241,900	875,600
Kwh per cu.yd.	4.29	5.08	5.45	7.19	5.38
Lift—ft.	96	213	74	83	....
Cut—ft.*	65	57	62	57	....
Pipeline—ft.	15,740	7,850	13,170	13,450	....

*From top of bank to bottom of cut.

Filling operations are similar to those followed at the dam (Figs. 11 and 12).

A new type of wastewater spillway is used at the dike (Fig. 13). The upper part is similar to the stepped sluiceways in use at the dam, but the lower part ends in a wooden box sump, from which lead two 28-in. waste lines. The lines discharge far down a coulee, splitting into four smaller lines for no reason except that the small pipe was available and not in use elsewhere. Plans are being made to convert the two wasteways at the dam into this type, which would permit start or removal of the lower portions of those wasteways just prior to diversion of the river.

Closure next year

It is expected that the diversion tunnels will be completed by early summer of next year, and the closure of the center section of the dam will then get under way. With four dredges pumping into this one section, the fill will rise about 1 ft. per day. Studies are now being made to determine the best type of core pool wasteway to use in the closure section and for the remainder of the dam.

Railroad access to the upstream side of the dam is now provided by a line extended around from the spillway. Spur tracks have been provided from the main line at elevations of the four 30-ft. berms on the upstream face of the dam and to the crest near the tunnel control shafts. These spurs will be extended on the berms when it is time to place the riprap on the fill slope.

Direction

The dam and dike are being built by the day labor method under the general direction of Lt.-Col. Thos. B. Larkin, district engineer. Major Clark Kittrell is in charge of operations with Capt. D. A. D. Ogden in direct charge of dam construction. C. S. Smith is superintendent of construction.

Test Pit Exploration Kit for Foundation Study

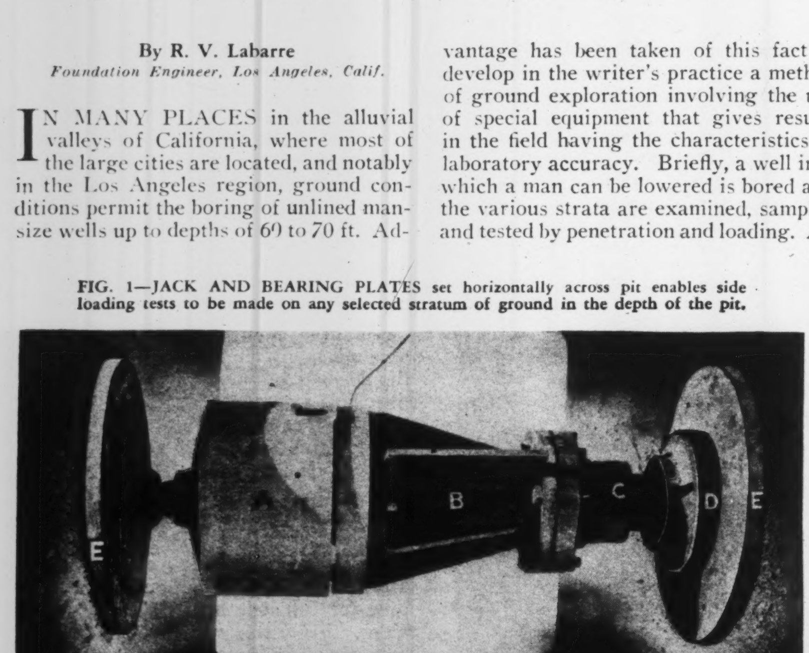
Apparatus for visual study, core sampling, peneration and loading tests for successive soil strata to depths up to 70 ft.

By R. V. Labarre

Foundation Engineer, Los Angeles, Calif.

IN MANY PLACES in the alluvial valleys of California, where most of the large cities are located, and notably in the Los Angeles region, ground conditions permit the boring of unlined man-size wells up to depths of 60 to 70 ft. Advantage has been taken of this fact to develop in the writer's practice a method of ground exploration involving the use of special equipment that gives results in the field having the characteristics of laboratory accuracy. Briefly, a well into which a man can be lowered is bored and the various strata are examined, sampled and tested by penetration and loading. An account of methods and equipment employed by the writer in testing soils and of some typical records is the purpose of this article.

FIG. 1—JACK AND BEARING PLATES set horizontally across pit enables side loading tests to be made on any selected stratum of ground in the depth of the pit.



Laboratory soil tests have been developed to very exact control and refinement of measurement and theory has been successfully developed to enable the prediction of large-scale soil behavior from measurements made in the laboratory upon small samples. To date, however, these laboratory tests have been confined in their usefulness and application to the analysis of the settlements of buildings founded upon thick beds of uniform material, usually sand and clay. The difficulty of applying and interpreting these tests becomes too great when the building site is underlaid by irregular lenses of varying and diverse soils, particularly when these lenses are composed largely of the loamy soils whose mechanics are not as yet well understood. The laboratory quantities may be measured with the same accuracy, but because of variation and discontinuity in strata, the utilization of these tests for predicting the behavior of the composite soil structure becomes too complicated and subject to even graver errors of interpretation than a field load test.

Peculiar subsurface conditions

In Southern California, many of the conditions encountered are peculiar to this region, which is semi-arid and characterized by a complex fault system, marked seismic activity and deep intermountain basins filled with poorly sorted and unconsolidated alluviums in which the water table varies widely. Field studies and explorations have often disclosed marked variations in soil structure between adjacent sites and not uncommonly within the boundaries of a single site.

These geological peculiarities greatly influence the seismic hazard of any particular locality, and the risk factors, such as the proximity of an active fault, and the geologic and foundation characteristics peculiar to the site, cannot be properly evaluated, nor can the type of structure or the minimum horizontal force which should be provided for in the structure be determined without the data furnished by a geologic survey of the particular area. Experience with such conditions has led to the practice of minimizing the amount of laboratory work and doing as much of it in the field as conditions justify.

A simple field method

Exploration shafts 2 to 3 ft. in diameter are drilled with a mobile power auger. These shafts are large enough to admit a trained observer for the purpose of examining the ~~subsurface strata~~ in place for type and character of the stratification and for immediate sampling and investigation of the materials observed. Nearly all of the materials encountered in this region are successfully drilled by the equipment indicated, and shafts of this size, where water conditions permit, stand safely without casing. In some cases shafts 60 to 70 ft. in depth have been successfully examined, logged and sampled. The cost of these borings compares favorably with hand auger borings.

Comparative hardness of the strata penetrated is measured with a penetrometer shown by Fig. 2. This instrument records the pressure in pounds required to force a needle or punch of determined size a given distance into the soil, and furnishes a rapid and accurate measure of the comparative variations in strata hardness. A graph of soil hardness is plotted on Fig. 3. This plate clearly shows the wealth of accurate data secured by this means of exploration. The sub-surface section shown is a summation of the data furnished by the geological survey of the site and the log of borings.

Samples are taken by the observer in the shaft at any points required by means of a sampler forced into the soil with a small hydraulic jack. The component parts of this sampler are shown in Fig. 2. The cutting edge of the sampler is case hardened and the inner chamber ground to a size 0.002 smaller than the chamfered brass rings assembled in the recessed part of the tool. The capacity is six rings 1 in. in height and 2.52 in. in diameter. After the sampler is withdrawn the cap is unscrewed, the rings and contained sample are slipped out and the sample is cut to suit the required test. All laboratory equipment is built to accommodate these ring samples.

In Fig. 2 is also shown the portable field shear test apparatus. A 3-ring sample is placed in this machine under a confinement corresponding to its natural

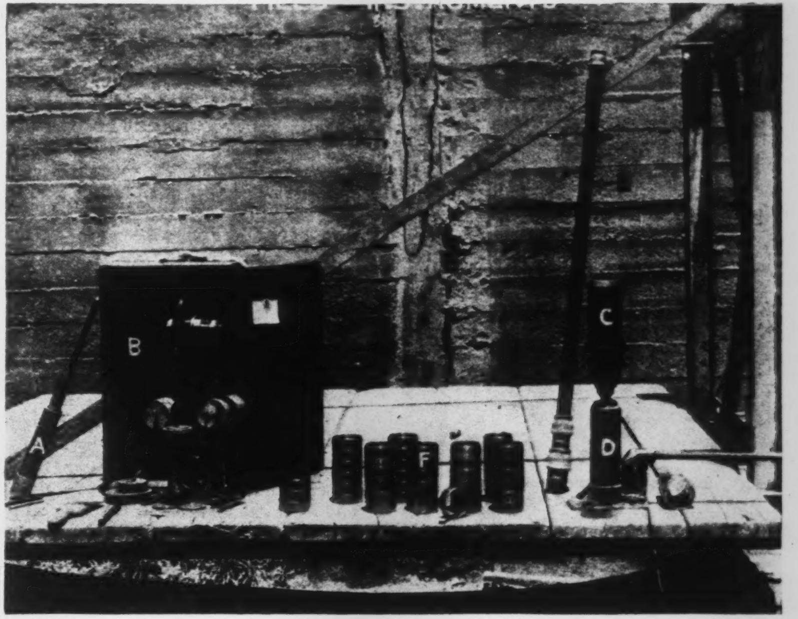
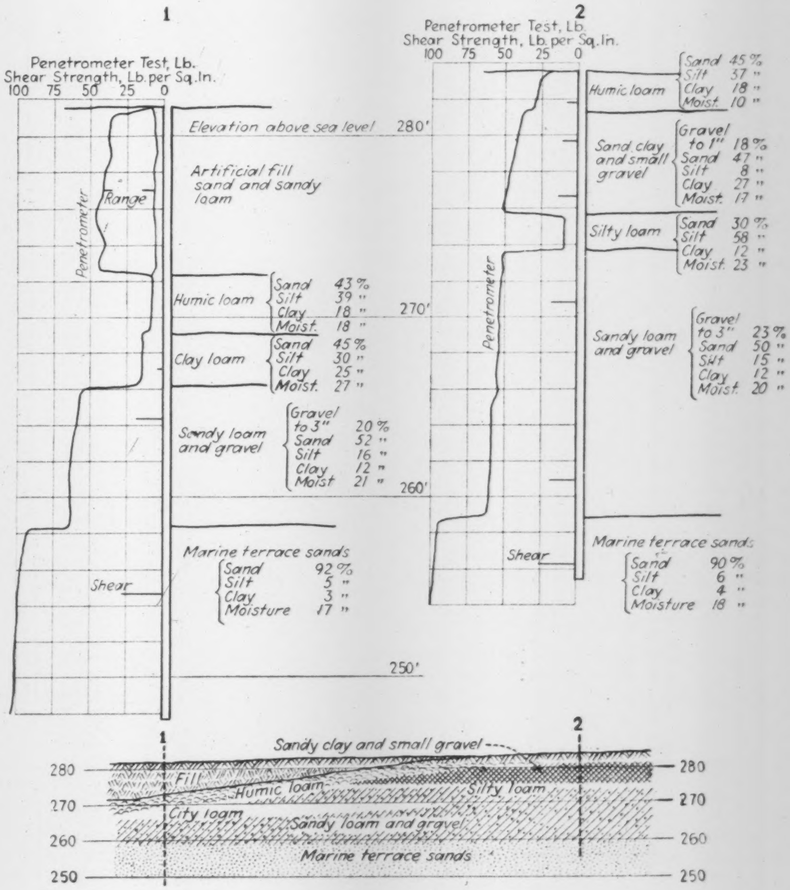


FIG. 2—A PORTABLE KIT OF TOOLS permits field tests and soil sampling for laboratory tests. A—penetrometer; B—shear machine and case; C—core sampler; D—core sampler jack; E—core sampler extension rod; F—sample containers.

FIG. 3—FROM PIT EXPLORATIONS, logs of penetrometer and shear tests and soil profiles are recorded in chart form.



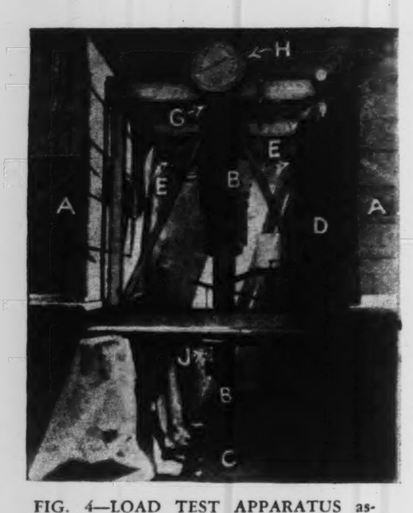


FIG. 4—LOAD TEST APPARATUS assembled from parts shown in Fig. 5 reacts against truss carrying sand bins for testing ground at any depth in the pit.

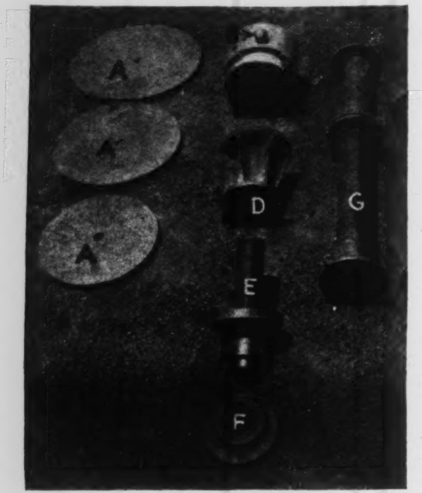


FIG. 5—ALL PARTS of the load-test apparatus can be handled by one man. A—bearing plates 4, 3, and 2 sq.ft. in area; B—gas-operated jack; D—jack pedestal; E—screw jack; F—bearing block; G—pipe column section.

surcharge and tested in double shear. Direct readings give the shear strength of the soil in pounds per square inch. The method of plotting these shear values is shown in Fig. 3.

Samples required for additional studies not readily performed in the field are sealed in the containing rings and sent to the laboratory maintained by this office.

Device for load tests

Load-settlement tests, where conditions indicate their value are made at such depths and locations as are indicated by preliminary borings, penetrometer readings and shear tests. The data obtained are plotted in the form of a load settlement curve. These tests are usually made on plates whose area ranges from 1 to 9 sq.ft., dependent on the characteristics of the strata investigated. The apparatus used has been especially designed to meet conditions of accuracy, portability, ease of erection and operation and to justify its more extensive use by materially reducing the field cost. The range and flexibility of this apparatus allows it use in vertical and horizontal tests (Figs. 1 and 4) in limited or cramped quarters. It has a capacity as shown of 50 tons. The jack proper has a capacity of 100 tons.

The component parts of this apparatus are: A bearing plate through which the load is distributed to the soil; a pneumatic jack which is the agency for applying the load, and a reaction for the jack. Various measuring devices and recording gages complete the assembly. Tests to be performed using this apparatus may be divided into three classes: (1) horizontal, where the ground itself furnishes the reaction; (2) vertical where the jack acts against a building or its footing, and (3) vertical where the reaction must be furnished by a loading frame.

In all tests, the bearing areas, which are completely interchangeable, consist of steel plates fabricated into circular disks ranging from one to nine square feet in area. (Fig. 5). This range of sizes permits the adaptation of the test to practically any condition or type of soil. The disks are pin centered, permitting an easy stacking of successively smaller plates above the test plate to provide increased section near the center and reduce to a minimum the deflection of the loaded area. The loading force is supplied by a gas-operated jack, nitrogen in commercial cylinders furnishing a convenient, safe and economical pressure source. The construction and performance of this jack permits a very definite and refined control of load. Regulating valves permit the admission or release of any desired amount of pressure to or from the jack and include provision to simulate earthquake loading. A dial pressure gage which, with the jack, has previously been calibrated, indicates the acting load at all times. The regulating valves are provided with a gas-control mechanism which maintains any previously set load by admitting additional gas to care for any opening of the jack in the course of settlement. Load is transmitted from the jack to the plates through a built-up column composed of a series of flanged steel pipe sections. (Fig. 5). Various lengths are available and with the aid of a screw jack of moderate extension a column of any length may be assembled. Jack and column assembly act as a unit and load is transmitted to both reaction and bearing plates through a spherical joint to insure axial loading.

In the third type of test mentioned, a loading frame is necessary. This consists of a demountable pin-connected truss or frame composed of eye-bars and standard light sections. This truss and all other parts of the equipment has been designed to carry a maximum load of 100,000 lb. and is best described as a weighted yoke against whose center the jack pushes upward. (Fig. 4). Plank boxes filled with earth weight the ends of this yoke. Any of the component parts of this apparatus may be handled by one man, and the entire unassembled apparatus is readily transported.

Load test procedure

From a knowledge of the sub-surface conditions at a site, obtained from borings and geological explorations, and an understanding of the conditions the proposed structure imposes, the critical strata and sections are determined and the test site selected. The test pit is then excavated to the desired depth either by hand or by a rotary caisson drill. If the location is an open lot or excavation, the bearing plates are carefully bedded at the bottom of the pit on undisturbed material, the loading frame is assembled at the surface and the boxes are filled with earth. After the large and immediate settlement of the surface material upon which the boxes rest is taken up, the jack and its loading column are inserted between plate and frame. The pressure system and gauges are connected to the jack and the test is ready for the application of load. (Fig. 4).

A base for measurement is established by fixing a bar across the shaft to which the instruments are connected; measurements are made with a dial micrometer gauge and also recorded automatically by magnifying levers on a revolving drum actuated by clockwork; settlements are given by the differential movement of the jack column and this bar.

Load increments are readily applied in any magnitude and as desired, and settlements recorded with time. Since no cumbersome weights must be handled, but only a small quantity of high pressure gas released, the device is particularly adapted to frequent rebounds of load to determine the elastic characteristics of the soil.

If the test is to be made below an existing building as in underpinning operations, or when alteration or removal of existing building and the construction of a larger structure is contemplated, the procedure is similar except the reaction is furnished by the building and no loading frame is necessary.

If the test is at a considerable depth and the stratum to be tested is of sufficient thickness to contain within itself the "bulb of pressure" developed beneath the test plate, the plate may be placed in a vertical plane and jacked horizontally against the soil. (Fig. 1.) In this case another plate either of similar or larger size is bedded against the diametrically opposite side of the shaft, the jack operating between them. Measurement of the movement of the plates is made from a base established between two points on the test shaft well situated 90 deg. of arc from the jack axis.

The simplicity, accuracy and range of this apparatus adapts it to various kinds of special tests. If heavier loads are required than one frame and jack can supply, as for tests on heavy piles, two frames and jacks may be set up beside each other and the load transmitted to the pile or other section by an I-beam.

The apparatus and methods developed, and here described, for making field tests have resulted in the accuracy of the laboratory being achieved in the field. The human element as affecting the interpretation of results is yet present and in the nature of things must remain. In this respect, laboratory and field tests are on a par, for to be of value the results must be interpreted by a trained foundation engineer thoroughly versed in soil mechanics. Each test should be used in its proper sphere.

Composite Trestle-Type Bridges on 100-Ft., Two-Column Bents

Timbers creosoted after framing—Split ring connectors effect saving—Relief labor employed on two duplicate viaducts

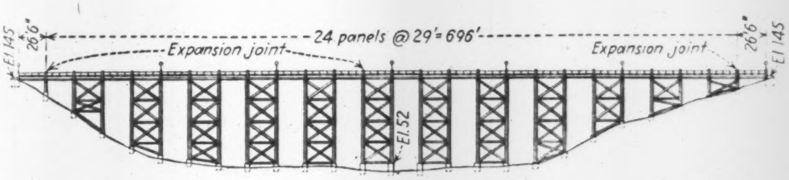
THE RECENTLY completed replacement of two old bridges on Eighth St. in Port Angeles, Wash., was made possible within the funds available by use of a trestle-type design with a composite deck in which a concrete roadway slab was bonded to timber stringers by means of daps and spikes. The bridges are of almost identical dimensions, each has a total length of 755 ft., made up of 26 panels of 29 ft. each. The maximum height is 100 ft. and each structure has a 24-ft. clear roadway with a 4-ft. sidewalk on both sides. The design is for a loading of H20. The structures are 900 ft. apart; one spans Valley Creek and the other Tumwater Creek. After timbers had been framed, creosoted and delivered to the job erection started Feb. 1; despite a loss of two weeks as a result of bad weather the timber erection was completed May 13. Opening to traffic was scheduled for Aug. 1.

The original plans were for ordinary trestle construction using creosoted piles in five-post bents 19 ft. apart. The piles were to range up to 106 ft. in length. The framing was to be done in Port Angeles, thus requiring a round trip to Seattle for creosote treatment. Cost estimates showed this plan to exceed the money available by some $10,000. Another factor was the shortage of skilled labor on relief at Port Angeles and surplus of skilled labor on Seattle relief rolls.

At this juncture Conrad O. Mannes, who had been making an analysis of the projects for the W.E.R.A. made a suggestion. His own comment on the suggestion is as follows: "I saw an article in *Engineering News-Record* (Oct. 4, 1934, p. 429) describing two viaducts of 48-ft. roadway built on Fourth Ave., Portland, Ore. They were of timber trestle type but had composite concrete deck and wood stringers with 29-ft. spans, three-column bents and tower construction. This looked like an economical design for Port Angeles. I made a preliminary check on the basis of two-column bents and a 24-ft. roadway and found a considerable saving. A. M. Young, district engineer, W.E.R.A., at once saw the advantage of using skilled labor on Seattle relief rolls for framing the timbers and common labor at Port Angeles for erection. Detailed plans of the Portland structures were obtained for study and then we started on what proved to be a novel design in bridge building. Many interesting problems had to be solved before the plan actually took shape."

As the Port Angeles bridges were to have only two-column bents, the columns were battered. Hence if steel gusset plates had been used, as was the case on the Portland viaducts, they would have been relatively large and costly. Instead 4-in. split ring connectors were employed throughout, a total of 8,000 were put into the two bridges. These were an important factor in the economical design.

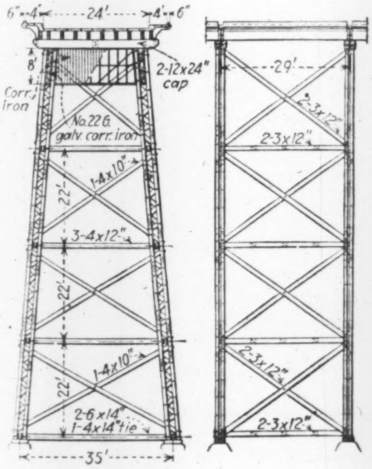
The deck construction is similar to that of the Portland viaducts; that is, timber stringers dapped on their upper edges and provided with projecting spikes to form a bond with the reinforced concrete deck slab. This is virtually T-beam construction with a span of 29 ft. center to center of bents. On the Port Angeles structures open lattice work was used on the columns as this provided adequate strength and gave the structure an attractive appearance. The structures were laid out on vertical curves 600 ft. long, thus leaving a short tangent at either end to avoid reverse curves in the approaches. The elevation at the center of each bridge is 1.135 ft. higher than at the ends, thus providing for drainage and camber.



TUMWATER CREEK BRIDGE, an elevation showing the type of tower construction used in both bridges.

ERECTING TRESTLE BENTS of one of the rebuilt bridges at Port Angeles. The upper half of a bent is being put in place for bolting to that part of the tower already erected.

TYPICAL BENT SHOWING bracing, batter-post construction and fire curtain.



A notable feature of the designs was that they were detailed and articulated so any individual member, even the stringers supporting the concrete slab, can be removed and replaced without disturbance of adjacent members. To facilitate this the maximum length of any member was kept down to 42 ft. This feature is expected to materially increase the effective life of the structures.

About 800,000 ft. b.m. of Douglas fir timber was used on the two structures. Creosoting specifications called for the use of a mixture of 50 per cent creosote and 50 per cent mineral oil. This combination materially decreased the cost of treatment and still gave the timber an estimated life of 35 years without allowance for renewals such as are possible under the special design.

Fire draft curtains, put in at 150-ft. intervals, have a design intended to protect both sides of the bent. They consist of 22-gage galvanized, corrugated iron on wood framework extending 8 ft. below the caps. The metal sheathing covers the bottom as well as all sides of the curtain.

The bents were erected by an electric stiff-legged derrick using a 76-ft. boom. This derrick was advanced along the top of the structure as erection proceeded. Supplementing the derrick was a steam winch on the ground mounted on caterpillar treads. Although the timber had been framed some months previously in Seattle and had been piled in the open until creosoted, all members fitted together properly and any warping or twisting that had developed was straightened without difficulty in the erection. Erection was completed without an accident on either job.

Financing of the structure was a joint venture as follows: city bond issue, $5[illegible],000; state grant, $26,140; PWA grant, $53,600. Plans were prepared under supervision of A. M. Young, W.E.R.A., district engineer; Conrad O. Mann[illegible], construction engineer, W.E.R.A., made the engineering studies. Contract for erection was let under PWA regulations; T. J. Murphy, PWA resident engineering inspector; H. E. Dodge is city engineer of Port Angeles. Contract for demolition of the old bridges and erection of the new was held by the Angeles Gravel & Supply Co. W. H. Rodgers supervised the timber framing in Seattle as well as erection at the site.

Graphical Aid in the Solution of Meyer's Evaporation Formula

By A. S. Levens
*Assistant Professor, University of Minnesota
Minneapolis, Minn.*

THE ACCOMPANYING alignment chart has been prepared to provide a quick and accurate solution of Meyer's evaporation formula,

$$E = 15\,(V - v)\left(1 + \frac{W}{10}\right),\ \text{where}$$

E = evaporation in inches per thirty-day month;

V = maximum vapor pressure in inches mercury corresponding to monthly mean air temperatures observed by Weather Bureau at nearby station;

v = actual pressure of vapor in air based upon Weather Bureau determinations of monthly mean air temperature and relative humidity at nearby stations;

W = monthly mean wind velocity in mi. per hr. as observed by Weather Bureau at nearby stations, about 30 ft. above general level of surrounding country or roofs of city buildings.

The above equation applies to small bodies of shallow water. For computing evaporation from large bodies of deep water, it is suggested that

V = maximum vapor pressure in inches mercury corresponding to water temperature instead of air temperature;

v = actual pressure of vapor in air about 30 ft. above water surface.

The chart makes it possible to solve for "E" very quickly. It obviates the necessity of (1) finding the value of "V" which corresponds to the monthly mean temperature; (2) computing "v" from "V" and relative humidity; (3) finding the difference between "V" and "v"; (4) computing $\left(1 + \frac{W}{10}\right)$ and (5) calculating the value of "E".

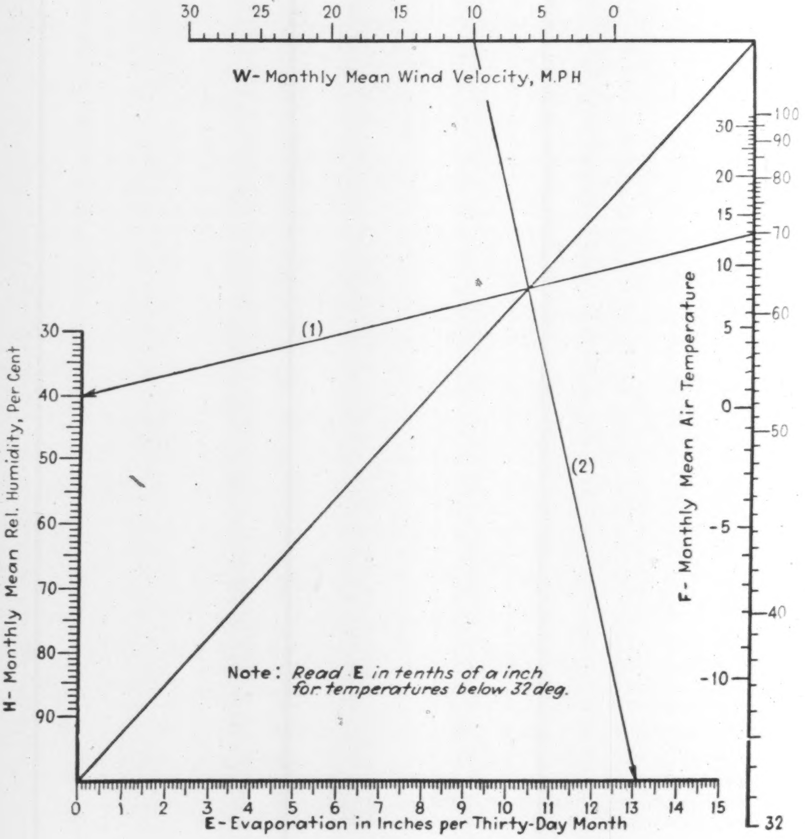
When the chart is used it is only necessary to know the

(1) monthly mean air temperature,

(2) monthly mean wind velocity,

(3) monthly mean relative humidity.

For example, in order to find the evaporation during a period when the temperature was 70 deg. F., the relative humidity 40 per cent and the wind velocity 10 m.p.h.: Join point 40 on the "H" scale with 70 on the "F" scale. Locate the intersection of this line with the diagonal. Join this point with point 10 on the "W" scale and read 13.15 inches on the "E" scale.



Note: *Read* **E** *in tenths of a inch for temperatures below 32 deg.*

MOSQUITO CONTROL ENGINEERING II

Mosquitoes: Species and Habits

Before measures of eradication can be successfully applied it is essential to understand the biology of mosquitoes — Such knowledge will reveal their vulnerability and thus indicate methods for effective control

By
DR. THOMAS J. HEADLEE

State Entomologist
N. J. Agricultural Experiment Station
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IN ANY PROJECT for mosquito control it is proposed to do certain things which will result in such reduced numbers of mosquitoes that the population in the protected district shall be free from further annoyance and danger from disease transmission. Past experience in mosquito control indicates that no project can be successfully prosecuted unless the engineer in charge first provides himself with knowledge of the species with which he has to deal and the habits peculiar to each species under differing conditions.

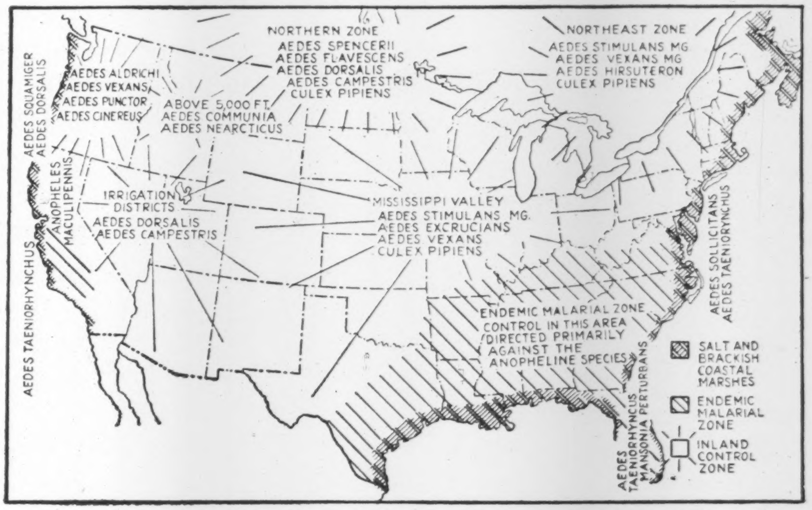

NORTHERN ZONE
AEDES SPENCERII
AEDES FLAVESCENS
AEDES DORSALIS
AEDES CAMPESTRIS
CULEX PIPIENS
NORTHEAST ZONE
AEDES STIMULANS MG
AEDES VEXANS MG
AEDES HIRSUTERON
CULEX PIPIENS
AEDES SQUAMIGER
AEDES DORSALIS
AEDES ALDRICHI
AEDES VEXANS
AEDES PUNCTOR
AEDES CINEREUS
ABOVE 5,000 FT.
AEDES COMMUNIA
AEDES NEARCTICUS
ANOPHELES MACULIPENNIS
IRRIGATION DISTRICTS
AEDES DORSALIS
AEDES CAMPESTRIS
MISSISSIPPI VALLEY
AEDES STIMULANS MG.
AEDES EXCRUCIANS
AEDES VEXANS
CULEX PIPIENS
AEDES TAENIORHYNCHUS
ENDEMIC MALARIAL ZONE
CONTROL IN THIS AREA DIRECTED PRIMARILY AGAINST THE ANOPHELINE SPECIES
AEDES SOLLICITANS
AEDES TAENIORYNCHUS
MANSONIA PERTURBANS
AEDES TAENIORHYNCUS
SALT AND BRACKISH COASTAL MARSHES
ENDEMIC MALARIAL ZONE
INLAND CONTROL ZONE

FIG. 1—DOMINANT MOSQUITO species and their general distribution.

There is an enormous amount of variation in the habits, life history and activity of different species of mosquitoes. Some mosquitoes, under favorable conditions, will move as far as 65 miles from the places where they developed, others will move only a few hundred yards. Several varieties do not bite man and domestic animals. Some species lay their eggs upon moist soil, others only on water. During the winter months particular types survive in the egg stage, others in the adult stage and still others in the larval stage. Depending on the species, the selected medium for breeding purposes may be fresh, salt or polluted water. Finally, only certain varieties carry disease—others are classified as pests.

In order to provide effective measures of control it is of utmost importance therefore, to know the different species and their habits under varying environmental conditions.

Principal mosquito species

The mosquito is a representative of the insect family known as Culicidæ and consists of two well-defined sub-divisions, the Chaoborinæ and the Culicinæ. The members of the Chaoborinæ are readily separated by the fact that the mouth parts of the females are not fitted for piercing, while in the Culicinæ the mouth parts of the females are elongated and fitted for piercing and sucking. There is no practical interest in the Chaoborinæ or non-blood sucking forms. We, therefore, turn to the sub-division Culicinæ, all of which are known to suck blood or plant juices.

Under the Culicinæ in North America we find the genus *Anopheles* with 7 species, *Aedes* with 65 species, *Culex* with 11 species, *Theobaldia* with 6 species, *Psorophora* with 7 species, *Mansonia* with 1 species, *Uranotænia* with 2 species, *Orthopodemyia* with 1 species, *Megharinus* with 1 species and the genus *Wyeomyia* with 1 species, making a grand total of 102 species. Naturally only a relatively small proportion of the total number of North American species are of economic importance.

For the purpose of getting at a conception of how many species the mosquito control agent will have to deal with, the New Jersey situation may be taken as an example. There are now definite records of 35 different species of mosquitoes occurring in this state. Of these, 6 belong to the genus *Culex*, 17 are *Aedes*, 4 are *Psorophora*, 4 are *Anopheles* and one each of the genera *Mansonia*, *Wyeomyia*, *Uranotaenia* and *Orthopodomyia*. Of these species, 2 members of the genus *Culex*, namely *pipiens* and *territans*, 6 members of the genus *Aedes*, namely *vexans*, *cantator*, *stimulans*, *sollicitans*, *taeniorhynchus* and *canadensis*, 3 members of the genus *Anopheles*, namely *quadrimaculatus*, *punctipennis* and *crucians*, and one member of genus *Mansonia*, namely *perturbans*, are of great economic importance. This means that out of the 35 recognized species of mosquitoes in New Jersey, 12 are of real economic consideration.

There is a considerable body of information available concerning the more important species of mosquitoes. This is all classified under the scientific name of the species and the only way the mosquito fighter can find out what is known is first to determine the species with which he has to deal.

Life history and habits

Hence, any person who proposes to undertake mosquito control operations must learn a great deal about mosquitoes, be able to identify species and know how to look up the facts regardless of where they may be published. The absolutely necessary key to unlock available information concerning mosquito species is a knowledge of the scientific name.

All species of mosquitoes have four definite stages in their life cycle, the egg, the larva or wriggler, the pupa and the adult. If, by any practical means whatever, one of these stages can be eliminated the mosquito pest disappears from the area in which it has formerly been prevalent.

For New Jersey conditions it can be said that all economic members of the genus *Culex* and of the genus *Anopheles* pass the winter in an adult form and are notably decimated in number by the effect of low temperature and exposure. All important members of the genera *Aedes* and *Psorophora* pass the winter in the egg stage and they are not reduced to any considerable extent by low temperatures and exposure. The single representative of the genus *Mansonia* passes the winter in the larval stage. The single representative of the genus *Wyeomyia* passes the winter in the larval form. Because of this great degree of freedom from destructive winter effects, mosquitoes of the genera *Aedes* and *Psorophora* can get on the wing early in large broods while *Culex* and *Anopheles*, because of the winter decimation, require considerable time in the spring to build up to a point where they are numerous.

In general, the eggs of members of the genera *Aedes* and *Psorophora* are laid on moist soil rather than on water surface. Consequently, the rising and lowering of water level makes wide differences where the eggs are laid down. Any water level variation such as occurs regularly, as is the case with the tides, seems to afford the optimum conditions for the production of enormous broods. The eggs of the genera *Aedes* and *Psorophora* are resistant to dry weather, cold weather, wet weather, in fact to all the exigencies of climate. They may lie unhatched and will be viable in the soil for years. Then when the proper conditions arise they may hatch very promptly and develop enormous broods. It is, of course, rare that members of the genus *Psorophora* appear in large broods but that is because of the nature of their feeding habits.

The picture of the members of the genus *Aedes* runs somewhat as follows: Practically the entire surface of the salt marshes and of the exposed low-lying soil around bodies of water in the swamp areas as well as the bottoms and sides of temporary pools are seeded with the eggs of *Aedes*. These eggs will not hatch until covered with water of suitable temperature. On the salt marsh the hatching waters are brought in primarily by high tides. On the swamps and uplands and temporary pools the water originates in rainfall. Whether or not a great brood gets off is dependent on the length of time during which this water remains and gives to the wrigglers not only a food supply but opportunity to gather it.

The eggs of the members of the genera *Anopheles* and *Culex*, on the other hand, are laid on water surfaces which are selected by the female as suitable places—these may range from pure spring water to the vilest sewage. However,

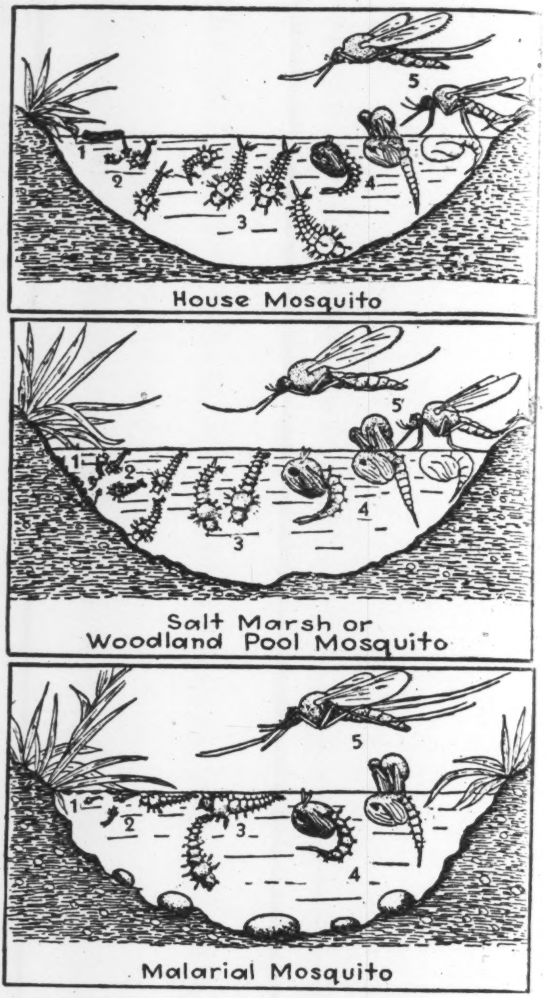


FIG. 2—LIFE HISTORY of three typical mosquito species: (1) Egg stage; (2) hatching larvae; (3) grown larvae or wrigglers; (4) pupæ stage; (5) adults emerging.

FIG. 3—Salt marshes are well seeded with mosquito eggs which hatch in warm weather when they are covered with water brought in by high tides.

FIG. 4—WOODLAND POOLS and swamps are generally the first bodies of water to produce mosquitoes in the spring of the year.

very few of these genera can breed on the salt marshes and they find conditions suitable only at the inner and fresher edges; breeding is especially influenced by rainfall.

Factors affecting development

Once the eggs of the mosquito have hatched the question of growth, persistence and development of the larva is a matter of food supply in the water in which it has been hatched and a continuance of this water. The food supply consists of unicellular animals and plants which are swept by the mouth brushes into the alimentary canal and then digested. Anything which prevents the development of this microscopic life results in the eventual destruction of these larvæ because they will die from starvation. The supply of unicellular animals and plants in water inhabited by mosquito larvæ is dependent upon the kind of bacterio-chemical digestion taking place in the pool bottom and that in turn depends upon the nature of the organic material in the bottom of the pool. Consequently, not all pools of water will support mosquito breeding.

Speed of larval development is dependent upon the temperature of the water. In midsummer *Culex* mosquitoes can complete the cycle in about a week, while in the spring or fall the same cycle may require three weeks. Among members of the genus *Aedes* the larvæ require from seven days in midsummer to four to five weeks to reach maturity.

The pupa, of course, takes no food but requires moisture for its persistence. This stage is usually short, varying from a day in midsummer to a week or ten days in cool temperatures of spring and fall.

When the adult emerges it sets out in search of food which, in many cases, is not blood at all but the juices of plants. Most mosquitoes can reproduce without blood. Furthermore, only the females can draw blood and the male must perforce live upon the juices of plants if he secures any food whatever.

Broods of adult females leave the places of breeding in search of food and also in search of new breeding spots. In this search these females may travel long distances. Salt marsh *Aedes* have been known to travel in New Jersey about 40 miles from a place of breeding, apparently going thither with the wind. On the other hand, *Aedes vexans* and *Culex pipiens* appear to travel distances only as great as ten miles and two and one-half miles respectively from the place of breeding, not going with the wind but rather in the direction of human population centers. The underlying causes of comparatively long distance movement of mosquitoes appears to be search for food and breeding places incident to the emergence of dense broods. The larger the emerging brood, the longer is apt to be the distance of migration. The activity thus initiated serves not only to continue the species where it is present already but to carry it to new breeding places.

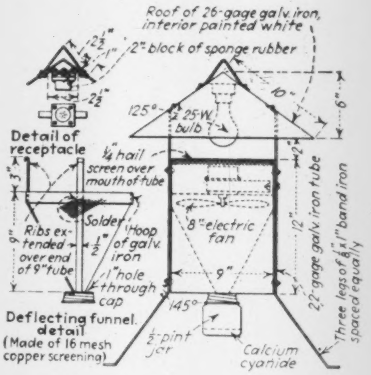
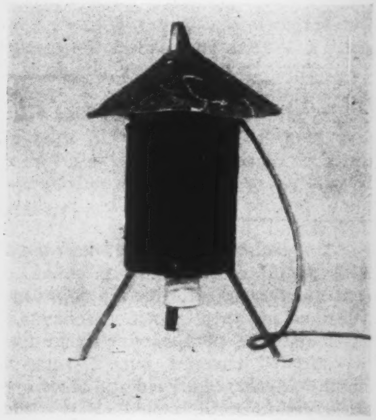

Roof of 26-gage galv. iron, interior painted white
2" block of sponge rubber
$2\frac{1}{2}$"
1"
$2\frac{1}{2}$"
10"
6"
125°
25-W. bulb
Detail of receptacle
¼" hail screen over mouth of tube
3"
9"
Solder
Ribs extended over end of 9" tube
½"
Hoop of galv. iron
1" hole through cap
2"
12"
8" electric fan
9"
22-gage galv. iron tube
Three legs of ⅛"x1" band iron spaced equally
145°
Deflecting funnel detail (Made of 16 mesh copper screening)
½-pint jar
Calcium cyanide

FIG. 5—MOSQUITO TRAP devised by N. J. Agricultural Experiment Station workers for survey purposes. At the bottom is the completed trap as it appears ready for use.



Summarizing, it can be said that mosquitoes are organisms carrying on their life cycles in hidden, obscure and little visited places. Some live in uninhabited wilds, some in sparsely inhabited places and others in densely populated cities. It is therefore, an exceedingly complex matter to ferret out enough of these breeding places to reduce the mosquito population to a point where human comfort is adequately preserved.

Determining kinds and number

Before intelligent mosquito control work can be undertaken in any area a knowledge of the mosquito fauna characteristic of that area, throughout at least one season, and better two or three, must be obtained. An experienced mosquito fighter by making a survey of a given area and its environs can arrive at a pretty shrewd judgment of what the mosquito fauna is apt to be, but it is also easy to gain incorrect impressions.

Assuming that the paramount importance attributed to a basic knowledge of the mosquito fauna of the proposed protected district is justified, the question then arises as to how this information can be secured. In New Jersey, electrically operated mosquito traps have been developed which, over a period of years, have demonstrated their ability to sample the mosquito fauna accurately. These traps are set up at the beginning of the season and automatically, at a predetermined hour every evening, go in operation and continue until the following morning. Each night's catch is then removed and identified. This process continues throughout the entire mosquito breeding season and the results are carefully analyzed.

The number of traps employed depends upon the size of the area, the distribution of the human population in it and the variations in the terrain. Since these are electrically operated devices and because the mosquito pest congregates close to human habitations, the most convenient location for the traps is in the yards of residences. If there are many towns and cities within the area all will have to be carefully spotted with traps and a large number will be required. If the proposed area includes sections separated by high hills or mountains, it will be necessary to trap the different sections, but always in places where people are living. The greater the number of traps employed the greater will be the accuracy of the survey but fairly satisfactory results may be obtained with only few traps.

It is, of course, advisable at the same time to run a survey of mosquito breeding places, collecting samples of breeding and determining the species concerned. When information from the trap records and from the survey of mosquito breeding places is correlated, it then becomes obvious whether migrants from outside the area form any particular important part of the mosquito fauna in the region under observation. If such should prove to be the case it then becomes necessary to extend the control operations beyond the confines of the proposed area to eliminate the possibility of migrational forms annulling the effect of control work within the area.

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In the third article to appear next week, general control technique and organization will be discussed by Russell W. Gies, sanitary engineer in charge of mosquito control, New York City Department of Health.

Precise Pumping Studies On Colorado River Aqueduct

Important contributions to pump technology have resulted from laboratory research carried out at Los Angeles, Calif., to develop the design of centrifugal pumps twice as large as present units in the United States

BEFORE designing the pumping equipment to be used on the Colorado River Aqueduct project of the Metropolitan Water District of Southern California, Los Angeles, elaborate investigations are being made in a special pump laboratory located at the California Institute of Technology. (*ENR* Nov. 1, 1934, p. 552). The special conditions in connection with pumping unusually large quantities of water against a total head of more than 1,600 ft. constituted problems in hydraulic and mechanical pump design for which no precedent existed. Some results of this important laboratory research and the conclusions reached, as presented by Max Spillmann, chief engineer of the Worthington Pump and Machinery Corp., at the Los Angeles meeting of the A.W.W.A., are reported in the following condensation of his paper.—EDITOR.

IN discussing the pumping plants of the Colorado River Aqueduct it is necessary to consider at first the conditions under which the equipment will be operated. Large quantities of water (600 sec.-ft. for the initial installation and 1,600 sec.-ft. when completed) will be pumped over a difference in elevation of about 1,360 ft., while an additional lift of about 300 ft. is required due to frictional resistance. Since these differences in elevation are distributed over considerable distance, five separate pumping plants will be used in series with several reservoirs of different capacities in between them.

The surface form of the terrain calls for two stations with about 440 ft. pump lift, two stations for about 300 ft. pump lift, and one station for approximately 150 ft. pump lift. It therefore is clear that especially for the high and medium head pumps the greatest care had to be exercised in developing the mechanical features of the pumping equipment. The large power consumption on the other hand made the efficiency of the pumps a feature of foremost importance because a difference in efficiency of 1 per cent on all pumps would correspond to a gain or loss of $40,000 per year in power costs.

Mechanical reliability sought

The greatest mechanical reliability, as well as high efficiency prompted the adoption of a vertical shaft arrangement for pumps and motors. One of the principal advantages of this arrangement lies in the possibility of placing the pumps low enough to obtain a positive head on the inlet side. Only with this arrangement could it be hoped to overcome even the highest pump lifts (440 ft.) in one stage without the danger of cavitation. However, the final answer to the qeustion of whether a single or two stage arrangement can be used in these plants required still further investigations. For this and other reasons, the Metropolitan Water District developed a special pump laboratory at the California Institute of Technology in Pasadena.

The pump engineer who has visited and seen this laboratory in operation must admit that it represents an almost revolutionary advance beyond standard pump testing practice. This is easy to understand for two different reasons: First, the district engineers and the research staff of the Institute could concentrate their efforts on the solution of a limited group of specific problems. In particular the pumps which had to be investigated covered but a relatively small field of speed, head and other operating conditions. Second, it was clear from the beginning that due to the tremendous importance of the efficiency of the final equipment considerable time, effort and money could be spent in order to obtain the greatest possible accuracy.

In view of well known practical difficulties the initial goal of obtaining an accuracy of 1/10th of 1 per cent for the efficiency results on the test pumps appeared to be almost too ambitious to permit a practically useful realization. Later results, however, have demonstrated that this goal has been reached.

Laboratory investigations

Some of the conclusions which the laboratory investigations revealed were as follows:

It was found that the pumping head per stage could readily be increased to 440 ft. or more without any sacrifice in efficiency, provided a corresponding inlet head is available. The single stage arrangement for all plants naturally led to a simpler arrangement of the pumping equipment than would have been possible with a two stage arrangement. Furthermore it was found that single suction pumps could be used successfully, allowing an arrangement whose mechanical advantage can be considered as well established through the experience with numerous water power installations.

Another important contribution of the laboratory was the measurement for the first time, of shaft deflections due to unbalanced radial forces acting on a pump runner in an open volute casing with the pump in operation. Promising attempts have also been made to explain the origin of these forces.

New form of pump casing

In discussing the mechanical features of the pump, attention must first be called to the fact that, according to present experience, the highest efficiency of centrifugal pumps is obtained by using volute casings without guide vanes. Hence a radial section through a pump casing in general will have the shape of a horseshoe open towards the center of the pump with the covers attached to the two ends of the horseshoe section. This form, which may be called the standard or classical one, therefore introduces a relatively high bending moment into the shell of the casing. For the high head pumps the Metropolitan Water District has specified a test pressure of 520 lb. per sq.in. and for this condition the conventional radial ribs used for the reinforcement of volute casings would have to be made extraordinarily heavy, thereby introducing appreciable foundry difficulties. Consequently a new form of design was developed resembling a short barrel closed by unusually strong covers. This form of design is not only sufficiently strong, but also very rigid, thus preventing distortion of the pump casing under the working pressure which might have proven fatal on account of the unusually close running clearances on the inside of the pump.

Record size pumps

Numerical data concerning the high head pumps are as follows:

The total head for these plants is about 440 ft. and the capacity per pump is 200 sec.-ft., or about 90,000 g.p.m. The speed of rotation is 450 r.p.m. and the resulting specific speed is:

$$N_s = \frac{rpm \times gpm}{H^{3/4}} = 13{,}300$$

or 63 if the capacity is expressed in sec.-ft. The power consumption of one unit amounts to about 12,000 hp. which is nearly twice as much as that of the most powerful pump which has been built up to now in the United States. In Europe, pumps with as much as 35,000 hp. are in operation, but these are equipped with guide vanes resembling those of high head water turbine installations.

Due to the high head requiring high water velocities inside of the pump the physical dimensions of the latter are not excessively great. The discharge diameter of the pump casing is 42 in., but adjoining diffuser gradually increases this diameter to 60 in. The inlet diameter decreases from 66 in. in the horizontal portion of the suction pipe to about 50 in. at the pump inlet. Laboratory experiments are still in progress to determine the best shape of the suction elbow.

The problem of the casing design is most readily understood by considering the total pressure, which in ordinary volute casings tends to open the horseshoe sections of the volute, but in this case is taken up by the bottom, the top cover, the outer shell and the ribs of the casing. This force normally amounts to about 2,500,000 lb., but temporarily may assume much higher values. Under the best pressure of 520 lb. per sq.in. this force increases to about 6,000,000 lb. The casing to withstand this load has a weight of approximately 66,000 lb. including the top cover which is integral with the bearing bracket.

In addition to the casing the dimensions of the shaft required special consideration. The diameter of the pump shaft is 22 in. at the bearing, gradually tapering down towards the impeller, while the diameter of the lower part of the intermediate shaft is increased to 24 in. The sole reason for this unusually heavy shaft is, of course, its rigidity, since the working stresses remain quite low under all operating conditions. It is possible that in this respect the aqueduct pumps will constitute a precedent for future designs. The shaft is a carbon steel forging with rigid flange couplings and has a central bore of 6 in. in diameter running through its entire length.

The impeller is about 85 in. in diameter and weighs about 7,000 lb. It is made from bronze consisting of 89 per cent copper and 11 per cent phosphorized tin, an alloy which is particularly suited for this purpose. The wearing ring clearance will be approximately 30/1,000 in. in diameter and can be checked through inspection holes.

Letters to the Editor

Nailed Timber Centering

Sir—In the issue of April 16, 1936, in the article *Nailed Timber Centering for Concrete Arch Bridges in Russia,* there is evidently a mistake in the conversion of kg/cm² to lbs./sq.in. as a stress of 110 kg/cm² is equal to a stress of only 1,565 lbs./sq.in. instead of the 15,650 lbs./sq.in. as given in the article. It is on page 564 and under the formula—

$$T_1 = 0.8d^2N_1 \times N^2$$

R. S. Soulé, Jr.

Shreveport, La.
June 25, 1936.

Reinforced-Concrete Beam with End Thrust

Sir—The article by Harry E. Eckles published in your issue of April 30, 1936, is, I believe, based on incorrect assumptions, and in view of the importance of the problem and the painstaking effort Mr. Eckles has made in his analysis and preparation of diagrams I should like to point out wherein his error lies, and to present a very simple method of analysis based on the usual assumptions.

Mr. Eckles concludes from his study of the matter that the assumption of plane sections does not hold in this case. It appears to me, however, that the assumption that plane sections before bending remain plane after bending is just as valid for a beam subjected to a thrust as for the ordinary beam subjected to bending moment only. Consider the case of a concrete strut without reinforcement, subjected to an eccentric load parallel to the axis but eccentric by an amount greater than one-sixth of the depth. Neglecting the tensile strength of the concrete, the beam will be cracked at all sections to a depth easily calculated on the theory of straight-line variation of the compressive stresses; each plane section through the beam will still be a plane section, and the distortion in any short length of the beam will be proportional to the distance from the plane of zero stress.

In the ordinary beam subjected to shearing stresses as well as bending stresses, plane sections become slightly curved due to the action of shearing stresses, but both theory and tests show that such curvature is too small to need consideration, and therefore the usual theory of plane sections is substantially correct. In the case of the strut here considered, shearing stresses are practically zero, so that no consideration whatever need be given to curved sections. It seems therefore that the so-called "conventional method of analysis," assuming plane sections, is valid in the problem discussed by Mr. Eckles. If not, then all the formulas and diagrams universally employed in arch analysis are wrong.

In Mr. Eckles' article there are two points where his assumptions have, I think, led to incorrect conclusions. The first is his development of eq. (5), apparently showing the error of the usual process. This equation is based on the use of the value of p obtained for the ordinary beam with bending moment only. This value of p is obviously incorrect, as the true value in this case is equal to that obtained from the bending-moment formula reduced by the amount of steel represented by the direct stress, or by $\frac{W}{f_s bd}$. If this modified value be used in eq. (4) there would result simple equality, as can be easily shown. Combining eqs. (3) and (4) we have

$$p = k_1 f_c / 2 f_s$$

which is the usual value. The net value of p is, then,

$$k_1 f_c / 2 f_s - \frac{W}{f_s \, bd}$$

and substituting this in eq. (2) we have the obvious equality

$$-W + 0.5 \; bd \; f_s \; k_1 = 0.5 \; bd \; f_s \; k_1 - W$$

This equality renders invalid his conclusion based on eq. (5).

Again, where Mr. Eckles states that direct compression adds to the compressive stress in the concrete but produces no change in the unit-stress in the steel, he is wrong. Such compression acts on both concrete and steel, increasing the compression in the former and reducing the tension in the latter.

The problem presented by Mr. Eckles is very readily solved, without diagrams, by the usual conventional methods. Take his own problem, wherein the bending moment about the tensile steel is 990,000 in.-lb., and the direct force is 30,000 lb. The beam may be designed as an ordinary rectangular beam for a bending moment of 990,000 in.-lb. and then the resulting steel area reduced by an amount equal to $\frac{30{,}000}{f_s}$. For $f_c = 650$ and $f_s = 16{,}000$ we have, from the usual tables or diagrams, $M/bd^2 = K = 107$ and $p = 0.0077$. Then $bd^2 = 990{,}000/107 = 9250$, and $b = 9250/27^2 = 12.67$ in.; $A = 0.0077 \times 27 \times 12.67 = 2.64$ sq.in.; net area of steel $= 2.64 - 30{,}000/16{,}000 = 0.77$ sq.in. If a width of 13 in. be selected then $M/bd^2 = \frac{990{,}000}{27^2 \times 13} = 104.3$, and for $f_s = 16{,}000$ the value of f_c will be 630 and $p = 0.0074$.

For the analysis of a given beam, which is also a common problem, the method of successive approximations may be used, or a diagram such as that published on page 434 of "Principles of Reinforced Concrete."

F. E. Turneaure,
Dean, College of Mechanics and Engineering,
University of Wisconsin.

May 21, 1936.
Madison, Wis.

Triborough Bridge Correction

In the list of engineers responsible for the Triborough Bridge at New York, published in *Engineering News-Record,* July 16, 1936, p. 101, the names of the consultants were omitted. The consulting staff consisted of Leon S. Moisseiff, consulting engineer; Daniel E. Moran, consulting engineer on foundations; Charles P. Berkey, consulting geologist, and Aymar Embury II, consulting architect.

ENGINEERING NEWS-RECORD

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August 6, 1936

Another Engineer

ENGINEERS cannot but find satisfaction in the appointment of Lieut. Col. Brehon B. Somervell as Works Progress Administrator at New York, the largest relief operation in the whole country. He is the second engineer to hold that position, William J. Wilgus having undertaken the reorganization of the WPA in September, 1934, and having continued as its head until May, 1935. Victor F. Ridder, the retiring administrator, succeeded Col. Wilgus. Like Col. Wilgus, Col. Somervell has made a name for himself as an executive. He comes to the difficult task at New York with fresh knowledge of relief operations gained in directing the Florida canal work and with a broad general knowledge of engineering that is a valuable asset in operations as diverse as those in providing work-relief around a great city. To make up for a freely acknowledged lack of such knowledge, his predecessor, Mr. Ridder, leaned heavily upon his chief engineer J. W. F. Bennett. This comment would not be complete did it not contain a word of commendation for Mr. Ridder, not only on the score of his ability as an executive, but because of his frankly expressed appreciation of the value of able engineering direction in relief operations. From his boyhood days Mr. Ridder was fortunate in being brought into close personal contact with the late Gustav Lindenthal, and the inspiration of that great engineer is clearly evident in his appreciation of the continuing need for the work of the engineer. At the time of his resignation he expressed the hope that he would be succeeded by an engineer.

Municipal Cleanliness

AMERICAN MUNICIPALITIES are showing a lively interest in the installation of sewage and refuse disposal facilities judging by comment received in the course of a recent editorial tour of communities in the Hudson Valley and southern New England. Municipal engineers and other public-works officials, queried on their plans for immediate and future improvements, in almost every case stated that they were primarily concerned with the problem of providing means for the purification of sewage and for the sanitary disposal of garbage and rubbish. This general appreciation and recognition of the importance of municipal cleanliness is a commendable attitude reflecting a progressive point of view. In part it is a result of extensive educational propaganda fostered by state boards of health but in greater measure it is a reflection of a country-wide municipal movement toward adoption of modern facilities to meet more exacting demands. The continued dumping of raw sewage into watercourses already degraded by pollution has long passed the point where it is condemned by esthetes alone—the practice is one that seriously menaces public health and it fails utterly to meet present day needs. The desire to substitute an hygenic and economical method of garbage disposal for the outworn practices of the past is finding expression in a quite general utilization of incineration. The merit of the method is well established, as witnessed by its growing popularity, and it will receive much more attention as further developments illustrate its applicability to low-cost power generation. The earnest and far-flung attack launched by our cities on their fundamentally important problems relating to wastes disposal is a gratifying indication of accelerated municipal progress.

Counterweighted Rigid Frames

WHAT IS BELIEVED to be the first counterweighted rigid-frame bridge to be constructed in this country has recently been opened to traffic near Columbus, Nebraska. This bridge, of concrete rib type, built by the Loup River Public Power District across its canal, also boasts the longest span, 109 ft., of any concrete rigid frame bridge in this country. There is a pertinent connection between these two facts, for counterweighting supplies an efficient method of reducing dead load moments at midspan and horizontal reactions at the base of the legs, both of which are factors that tend to limit span lengths. The world's longest rigid frame bridge, the Herval Bridge, of 224 ft. span, in Brazil, is of the counterweighted type. On the Columbus bridge, however, the counterweighting seems to have been handled better than on the Brazilian structure where the counterwieghts are suspended from hangers attached to the ends of the cantilever side spans. At Columbus, by casting the counterweights of solid concrete integral with the ribs, all special maintenance problems are eliminated and, at the same time, the ends of the cantilever are given a useful lateral rigidity. If counterweighting is to be effective in reducing midspan moments there must be freedom of movement at the base of the legs for which there are numerous hinge designs available. The counterweighted rigid frame would seem to warrant more consideration by bridge engineers. Its usefulness in permitting longer spans has been demonstrated. Careful study may reveal that it offers economies for locations where single short spans have heretofore been accepted as a matter of course.

To Avoid Confusion

ANY OBSERVER at the International Soil Mechanics Conference in June could not fail to be struck with the difficulties of creating a simple and clear terminology for a new art. Concepts and quantities not previously defined, or perhaps even recognized, easily become involved in confusion of name. New properties that come under observation are not seen in the same way by different people, and there is risk of talking at cross purposes through using the same terms. "Consistency," for example, is applied quite as loosely in the field of soils as in the field of mortar and concrete, where it has long been a source of misunderstanding and aimless discussion. Still more characteristic for the new science, however, is the term "consolidation," which is applied variously to the slow squeezing and drying out of clay strata under a superimposed load, to natural or artificial compaction of loose soil to form a seasoned roadbed, and to the artificial hardening of subsoil by grout injection or by chemical or physico-chemical treatment. A clear and understandable terminology is a real need in this new field—in fact an essential to smooth and unimpeded progress. It cannot be created by fiat, of course, or by formal standardization; this would be fettering and ineffective. But individual judgment and discrimination by all who work in the new field, to use terms in clearly defined ways and to avoid confusion with other meanings of one and the same term, can be definitely helpful. If with such effort there is also a conscious striving to avoid multiplication of new (and generally unnecessary) "coefficients" and "limits," much future confusion and misunderstanding may be avoided.

Ground Studies Mechanized

TO THE MANY READERS who recall the perfunctionary wash boring and loading tests of a former day of soil foundation investigation the elaborate exploratory outfits of the modern foundation engineer present a constant surprise. As an example the ground-study outfit described by Robert V. Labarre in this issue probably represents an extreme, but others not so far inferior are being used, particularly by European engineers. The significant fact about all these mechanical devices is that they are not something originated for an isolated expreriment but are regular working tools of the foundation engineer. The apparatus described by Mr. Labarre is equipment in regular use in his professional practice. The fine promise of increased use of such devices is that they afford an opportunity for establishing a basis for standardization. Combined with parallel laboratory determinations of soil properties, records will accumulate from which may be determined, in turn, uniform meanings of penetration and load tests in terms of shear and consolidation. It is in linking field and laboratory records that soil mechanics now has the most to gain and in this respect the practicing engineer has a very important duty to perform in aid of soil mechanics research.

Controlling Road Transport

CREATION by the New York legislature of a permanent traffic commission is a forward move in highway operation. One regrets that the merit of the action is dimmed by the mist of hesitancy in giving it force. As the commission exists the financing is grossly insufficient, administration is a part-time duty and control is limited to state roads, or about 14 per cent of the mileage of roads and streets in the state. Within its jurisdiction the commission has however all needed authority to set up traffic control facilities and regulations and to penalize their disregard.

In its intended purpose as a safety measure the creation of the New York traffic commission is only partly effective. About 86,000 of the 200,000 miles of New York roads and streets are under no additional control except as the commission can influence action by counties, towns and cities. The force of this influence is wholly a question of wise and vigorous administration—one that can give all its time and energy to the work and has ample working funds. The present commission qualifies in neither of these requirements. No question of competency of the commission members named by the legislature arises but each already has a full-time task as the administrative head of an important state bureau; his traffic-commission work must necessarily be a side issue even with the most conscientious determination on his part.

The commission act can be strengthened. The important step was taken when recognition was given in law to the fact that the state has a duty in operating as well as in the construction and upkeep of its public roads. Transport and public safety have reached conditions where this function can no longer be overlooked if movement of goods and people by highways is to reach the magnitudes of which it is capable.

Industrial Decentralization

OF THE STOCK terms that are met in every analysis of current economic conditions, none is more popular at the moment than Industrial Decentralization. The term is used in everyday talk as though it described something very real and definite, when actually there are relatively few examples of plants being moved to small agricultural towns far away from established industrial centers, such as the much heralded movement of garment workers from New York to Hightstown, N. J., that began in a small way this week.

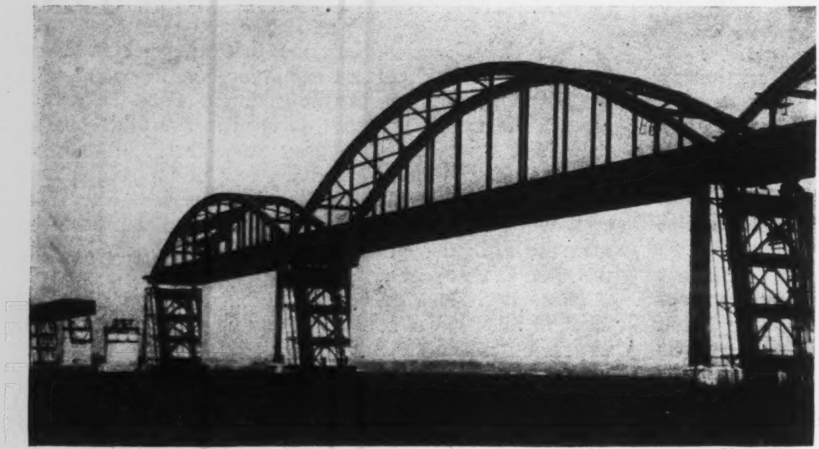
In such a sense industrial decentralization does not exist at all. What actually is taking place is industrial expansion within existing industrial areas. This naturally entails decentralization of a sort because suitable and cheap land is only available on the outskirts of the areas. Chicago, Pittsburgh, New York, Baltimore, Buffalo, Cleveland and numerous smaller large cities comprise the hubs of this decentralization. In an economic sense this development is of course important but it hardly threatens a change in the existing industrial order, a fear which is often expressed when industrial decentralization is discussed. Furthermore, it offers no threat to existing industrial areas. Quite the contrary.

One thing that it does mean, however, was brought out some months ago by N. H. Engle, assistant director of the Bureau of Foreign and Domestic Commerce in an address before a conference on low-cost housing at Pennsylvania State College. Decentralization, warned Mr. Engle, brings in its wake the problem of providing suitable housing facilities adjacent to the new factories, and this places a responsibility on the citizens of the community for sound planning if the creation of modern slums is to be avoided. Every city engineer knows that such decentralization or dispersion of factories gives rise to many problems beside housing, some corrollary to it, some separate. Sewage, water supply and transportation may be mentioned. Adequate zoning and effective street patterns are other requirements.

The problem of the satellite village was thoroughly bungled on the last opportunity, when speculative housing developments mushroomed in every direction from our large cities. In that instance the tail was allowed to wag the dog. There was no community planning at all. A checkerboard landscape of shoddy houses came into existence over some week-end, followed by the demand for water and sewer main extensions, paved roads and perhaps a trolley line or a rapid-transit railroad. These community facilities when built were hastily planned, and too often hastily built.

The rate with which industrial dispersion is now going on suggests a recurrence of the old evils unless the problem is taken in hand immediately. Industry, in moving to the outskirts, is evidently seeking escape from the unbearable conditions of its old surroundings. This suggests that industry's management is a powerful ally in establishing the planned modern community that we read so much about and seldom see. Town planners, city engineers, municipal officials and low-cost housing experts therefore are face to face with the opportunity that they have been waiting for.

CURRENT NEWS



Globe Photo

CHANNEL SPANS of the two-mile Danish bridge from New Zealand to Falster

Longest Bridge in Europe Nears Completion

The new railway and highway bridge across the Storstrommen, joining the Danish islands of Zealand (upon which Copenhagen is located) and Falster, is nearing the end of a three-year construction program. Its length of over two miles makes it the longest bridge in Europe.

The bridge will carry an 18½-ft. roadway, an 8-ft. sidewalk, and a single track railway. Rising from each end toward the center on a 0.7 per cent grade, a clear-headroom for shipping of about 85 ft. is provided at the three navigation spans. As shown in the attached view, these navigation spans are stiffened tied-arches. The center span is 450 ft. long and the two side spans 340 ft. The 47 approach spans, of the cantilever type with suspended spans and anchor arms located in alternate openings, are alternately 190 and 204 ft. long, and consist of two plate girders 12 ft. deep set 24 ft. apart.

The arch ribs of the navigation span are about 3 ft. deep. The stiffening girders, 12 ft. deep, are set about 40 ft. apart. Structural section hangers are used. Floor beams 4 ft. deep are framed into the stiffening girders at each hanger point.

The greater part of the 30,000 tons of steel required is of the high tensile type known as Chromador steel manufactured by Dorman Long & Co., Middlesbrough, England, who are also the fabricators and erectors. The cost, about $6,000,000, is borne by the Danish government.

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Correction

The news-item headed Contractor Held Liable for Wages on Subcontract on page 178 of the July 30, 1936, issue of *Engineering News-Record* stated, incorrectly, that the decision in question was handed down by the New York State Court of Appeals. The decision was handed down by the Appellate Division, Fourth Department.

Max C. Tyler Appointed Assistant Chief of Engineers

Max C. Tyler was appointed Assistant Chief of Engineers, U. S. Army, with the rank of brigadier general on July 30. The appointment was made by President Roosevelt on the recommendation of Secretary Dern and was authorized by law passed by the last Congress increasing the strength of the Corps of Engineers and providing for two assistants to the Chief of Engineers.

Gen. Tyler has been in charge of the river and harbor section in the office of the Chief of Engineers in Washington since April 20 this year. He had been division engineer of the Great Lakes Division at Cleveland from October, 1933, to April, 1936, and a member since July, 1934, of the Board of Engineers for Rivers and Harbors. General Tyler previously served as district engineer at various points. He was chief engineer of the Federal Power Commission from May, 1929, to June, 1931, and has been connected with many important river and harbor improvements, including the construction of Wilson Dam at Muscle Shoals, lower Mississippi River improvements, and engineering work on the Great Lakes. He is a graduate of West Point in the class of 1903. During the World War he served as executive officer and military advisor to the Director General of Military Railways.

◆

Two Highway Organizations Set Winter Meeting Dates

The date for the convention of the American Road Builders Association has been set as the week of January 11, 1937. The place where the convention is to be held has not been decided upon as yet.

The annual meeting of the Highway Research Board has been moved forward two weeks in order to avoid conflict with the meeting of the American Association of State Highway Officials, and will be held in Washington, Nov. 18-20, 1936.

Drought

Special Correspondence

Commercial and domestic water supplies in the drought areas throughout the Middle West continue to drop. Over broad regions stream flow is diminishing to lower levels than during the same season of 1930 and 1934.

Reports from district engineers of the U. S. Geologic survey on commercial and domestic water supply conditions of states in the drought area follow:

Arkansas

Although most streams in Arkansas are not as low as they were in 1934, in some sections springs and wells are lower than has ever been recorded. High temperatures and deficient rainfall have continued in most sections.

Indiana

Stream flow in Indiana continues to decrease and is now below that for the same period in 1934, although it is somewhat above the low flows that occurred later in the season in 1930 and 1934. Restrictions on the use of water and hauling of water for domestic and stock use continued in a few localities, but no serious shortages of water are known. Temperatures are moderate and good rains have fallen recently in the south central and northeastern parts of Indiana, greatly aiding in maintaining the stream flow.

Missouri

Flow of streams in the northern half of Missouri is still somewhat above the corresponding period of 1934 and appreciably above the minimum of 1934. Flow of streams in the southern part is about the same as during the corresponding period of 1934.

Minnesota

Water supply conditions in the Red River Valley in Minnesota are serious. Temperatures throughout the week in most sections have been normal. The few scattered showers have been so light that they have been of little benefit in the water situation.

Montana

A shortage of water for domestic use is now being experienced in the foot hills in the western portion of Montana. In general, conditions are somewhat more severe than during the previous week. The drought conditions are spreading further west. High temperatures throughout the drought area in Montana continued, with no precipitation over most of the area.

North Dakota

Municipal water supplies for cities in the Red River Valley in North Dakota have remained critical, and studies are being made for emergency measures looking toward relief of the situation. There has been little change in conditions in the rest of the state. The water supply situation in other parts of the state is still very critical. Most supplies, with the exception of those for Bismarck, Mandan and Fort Gates, are obtained from wells, which are showing marked effects of the drought. Valley City has partially overcome its problem by using the Cheyenne River, pumping the water to an old gravel pit.

The flow of the Missouri River on July 25 was 15,800 sec.-ft. as compared to 6,610 sec.-ft. for that date in 1934.

South Dakota

Water supplies in South Dakota are continuing to diminish, and all tributaries of the Missouri River are lower now than at the corresponding time in 1934. The flow of the White River on July 25 was only 1 sec.-ft., the lowest flow on record, as compared to 18 sec.-ft. at the same time in 1934.

Wyoming

Water supply conditions in Wyoming during the week became more acute. Although there have been a few rains, they have been too widely distributed to give much relief, and have not materially changed the general outlook.

Mississippi River

On July 31 the stage of the Mississippi River at Davenport, Ia., was within 0.2 ft. of equalling the all-time low for the navigation season. Flow was only 11,800 sec.-ft.; the normal discharge is 50,000 sec.-ft. Since the lowest stages of the river do not occur normally for another month, still lower stages are expected this year. Barge service on the Mississippi above Dubuque, Ia., has been temporarily abandoned by the Inland Waterways. At several places the present navigable channel depth is only 4.3 ft. None of the cities using the river for water supply have reported shortages. However, a number have been forced to limit the hours for sprinkling lawns because supplies are insufficient for possible peak loads.

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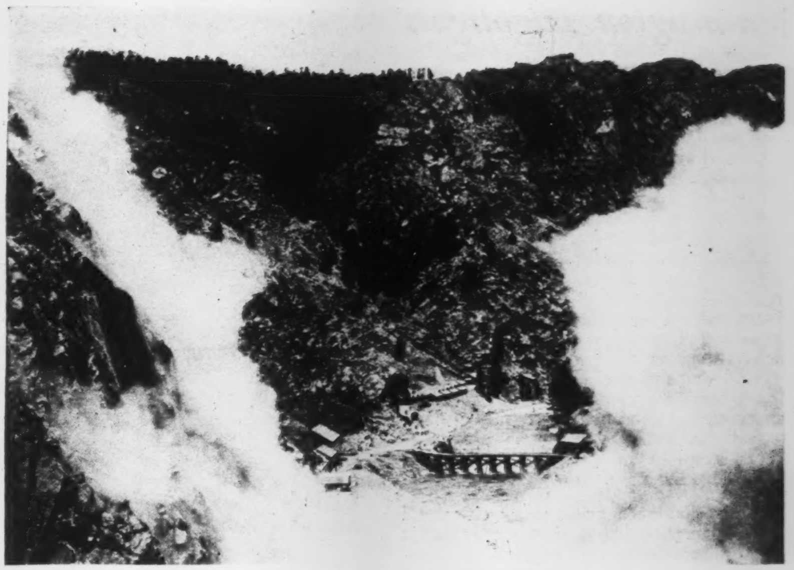
Seven-Mile Water Tunnel To Be Built at Baltimore

The city of Baltimore will advertise, early in September, for bids on the construction of the proposed Gunpowder Falls-Montebello tunnel connecting Loch Raven and Lake Montebello.

The conduit will be about 36,700 ft. long. Of this, 34,500 ft. will be in rock tunnel, and 2,200 ft. will be in open trench. The tunnel section will be 12 ft. inside diameter. About 12,000 ft. of the tunnel will be lined either with reinforced concrete pipe backed with concrete, or else with steel plate pipe backed with concrete or shotcrete and lined with shotcrete. The rest of the tunnel section will be lined either with shotcrete or with concrete; a small amount of reinforcement will be used in either case. The trench section will consist of 10 ft. steel-plate pipe with a bituminous lining and a concrete casing.

Bids for the construction work will be opened within 20 days after advertisement, and city officials suggest that prospective bidders visit the work before the bids are advertised.



WORK BEGUN ON SEMINOE DAM

Blasting the abutment for the Seminoe Dam on the North Platte River, Wyo., one of the two principal stuctures of the Casper-Alcova power and irrigation project. Smoke and dust frame the contractor's camp and the construction bridge.

Interstate Commission For Delaware River Basin

An interstate commission, including representatives from Pennsylvania, New Jersey, New York, and Delaware, is now at work in an attempt to coordinate federal, interstate, state and local activities in the development and control of the waters of the Delaware River basin.

The commission is composed of three commissioners on interstate cooperation and a member of the planning board from each of the three states of Pennsylvania, New Jersey, and New York. Three representatives have been appointed by the governor of Delaware, which state has neither a commission on interstate cooperation nor a planning board. Also on the commission are the regional representatives of the Council of State Governments and a technical water consultant appointed by the federal National Resources Committee.

Thomas A. Logue, secretary of Internal Affairs of Pennsylvania and chairman of the Interstate Commission, states: "The Interstate Commission on the Delaware River basin is not a study group; rather it is designed to put into effect a program which we hope will correlate and harmonize the viewpoints and recommendations contained in the various surveys and studies of the Delaware River which have been made from time to time during the past 50 years. Each of the four states concerned is officially participating in this work."

The first objective of the commission is to devise a program for the reduction of water pollution in the Delaware River and its tributary streams. A review of all water legislation in the four states involved is now being undertaken. A legislative program will be prepared for submission to the January, 1937, legislative sessions of the four states concerned with the waters of the Delaware.

E.C.P.D. Meeting Date Set

The date for the fourth annual meeting of the Engineers Council for Professional Development has been set as Tuesday, Oct. 6, 1936. It will be held in the Engineering Society Building, 29 West 39th St., New York City, and will occupy an entire day.

◆

SOCIETY CALENDAR

ILLUMINATING ENGINEERING SOCIETY, annual convention, Buffalo, N. Y., Aug. 31 to Sept. 3.

CANADIAN GOOD ROADS ASSOCIATION, annual meeting, Charlottetown, P.E.I., September 1-3.

THIRD WORLD POWER CONFERENCE, Washington, D. C., Sept. 7-12.

PUBLIC WORKS CONGRESS, joint meeting, American Society of Municipal Engineers and International Association of Public Works Officials, Toronto, Ont., September 28-30.

CANADIAN INSTITUTE OF SEWAGE AND SANITATION, Royal York Hotel, Toronto, Oct. 1-2, 1936.

INTERNATIONAL CONFERENCE ON STEEL CONSTRUCTION, Berlin, Germany, Oct. 2-7.

INTERNATIONAL ASSOCIATION FOR BRIDGE AND STRUCTURAL ENGINEERING, second congress, Berlin, Germany, Oct. 11.

AMERICAN WATER WORKS ASSOCIATION, Central States Section, Cleveland, Ohio, Aug. 19-21.

ENGINEERING INSTITUTE OF CANADA, maritime general professional meeting, St. Johns, N. B., Aug. 27-29.

AMERICAN WATER WORKS ASSOCIATION, Rocky Mountain Section, Denver, Colo., Sept. 21-23.

NEW ENGLAND WATER WORKS ASSOCIATION, annual convention, New York, N. Y., Sept. 22-25.

AMERICAN WATER WORKS ASSOCIATION, New York Section, fall meeting, Saratoga Springs, N. Y., Oct. 22-23, 1936.

EXAMINATIONS for registration as Engineers, Land Surveyors and Architects will be held as follows:

NEW MEXICO, examination for professional engineers and land surveyors, Santa Fe, Sept. 12.

Washington Highlights

By Paul Wooton

A list of flood control projects in New York and Pennsylvania, selected from those authorized by Congress in the Omnibus Bill last session, were to be submitted this week to President Roosevelt by Secretary Ickes, as chairman of the National Resources Committee. Preliminary work on such projects will be undertaken by WPA labor in anticipation of an appropriation by the next congress in January that will permit the award of contracts. The projects were selected by the water resources committee of the national resources committee in consultation with the corps of engineers, the soil conservation service and other federal agencies concerned in a coordinated program of flood control. A list of projects on which the army engineers have begun studies was published last week.

Elimination of a great many cities and towns from participation in the new public works program is seen by Secretary Ickes' as a result of President Roosevelt's insistence that 100 per cent relief labor shall be employed. If this requirement stands, said Secretary Ickes, it is "very problematical" whether a sufficient number of projects will qualify to absorb the $300,000,000 available for distribution as 45 per cent grants against their cost. Asked Aug. 4 whether there is any chance of a change in the regulation, Administrator Ickes' said that he had not talked with President Roosevelt since he left Washington and that it is up to him. PWA has submitted projects to the President for which varying percentages of relief labor are available, but none has been approved except for communities that have been able to certify that the project can be built entirely by labor drawn from WPA rolls. Administrator Ickes said that he would like very much to go ahead with more projects without further delay.

SLUM CLEARANCE IN NEW YORK CITY

Twelve blocks of Brooklyn slums have been cleared out to make way for PWA's Ten Eyck Houses development in New York City. The largest project on PWA's slum clearance program, Ten Eyck Houses will provide homes for 1,614 low income families. Construction is now under way. In the center of the site a school is being erected by the city to serve the new community. PWA allotted $12,783,000 for the project.

Bids were received on Aug. 4 for the construction of the superstructure of this project. Starrett Brothers and Eken, of New York, were low with a bid of $7,610,000.

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First Power from Norris Dam

Power was generated at TVA's Norris Dam on the Tennessee River, for the first time on July 28. Power is being sent over a 220-mile 154,000-volt line, to Wilson Dam on the Tennessee River in Alabama.

Revenue produced by the sale of power generated at the Wilson Dam of the Tennessee Valley Authority more than doubled during the last fiscal year. Gross revenues for the twelve months ending June 30, 1936, were $1,197,673, as compared to $595,188 for the year ending June 30, 1935.

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Flood Control Association Reorganizes

At a meeting on July 24, in Memphis, Tenn., the Mississippi Flood Control Association, an organization of representatives from levee and drainage districts in Missouri, Arkansas, Mississippi, and Louisiana, was dissolved, and a new association, the Mississippi Valley Flood Control Association, was organized.

Officers of the new association are: president, Senator John H. Overton of Louisiana; vice-presidents, Wade O. Martin, chairman of the Louisiana Public Service Commission; J. G. Burke, attorney for the Laconia Drainage and Levee District, and J. W. Bradford, president of the Yazoo-Mississippi Delta Levee District; secretary-treasurer, Fred E. Benneke, secretary of the National Rivers and Harbors Congress.

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Electric Power from Garbage At Providence R. I. Incinerator

Combining power generation with refuse disposal, the new 160 ton municipal incinerator plant for Providence, R. I., will be officially placed in operation on Aug. 10 with appropriate dedication ceremonies. The plant is designed for 24-hour continuous service and will develop sufficient power to operate the municipal sewage pumping and treatment works. The latter employs the activated sludge process and the high degree of clarification with this type of treatment makes it possible to return part of the effluent to be used as cooling water in the condensers of the turbine driven generators.

The new plant, based on studies conducted by Charles A. Maguire, commissioner of public works, and Philip J. Holton, Jr., mechanical engineer, contains a 5-cell Hiler incinerator so arranged that the hot gases resulting from the combustion of refuse are conducted to either one of two 700 hp. waste heat boilers. These boilers, operating at 200 lb. pressure, are connected with two turbine driven generators with a capacity of 1,250 kw. each. Units have been installed in duplicate in order to assure continuous operation. In addition, provisions have been made to fire the boilers by oil burners when the furnaces are temporarily out of service for relining.

No Hearing to be Held On Safety of New Croton Dam

The New York State Supreme Court has denied an application of Margaret Mayo, of Harmon, N. Y., for an order to compel Col. Frederick Stuart Greene, State Superintendent of Public Works, to hold public hearings on the question of whether or not Croton Dam is an unsafe structure.

Miss Mayo had requested Mr. Greene to hold a public hearing on the matter. He refused, declaring that it would be an unprecedented and unauthorized form of procedure and pointing out that he had no authority under the law to act in a judicial capacity in such matters.

In the course of the appeal to the higher court, it was alleged that the core of the dam is cracked; that the stone is disintegrating; and that the dam is leaking dangerously.

Mr. Greene introduced engineering testimony that the dam is in good condition and safe, and that such leakage as exists is negligible in quantity. Among the engineers testifying to the safety of the dam were: J. F. Bixby, district engineer of the state Department of Public Works; Thomas F. Farrell, chief engineer of the state Department of Public Works; and Maurice P. Davidson, commissioner of the department of water supply of New York.

New Croton Dam is a part of the water supply system of the city of New York; it is 200 ft. high, 2,016 ft. long, and impounds more than 30 billion gallons of water. It was built in 1892 by the New York Aqueduct Commission; it consisted at that time of three sections: a masonry overflow weir, a masonry dam section, and an earth dam section with masonry core wall. Changes in the structure since that time have eliminated most of the earth dam section, substituting masonry construction.

Jersey Water Boards Reach Agreement

The two boards responsible for northern New Jersey water development, the New Jersey State Water Power Commission and the North Jersey District Water Supply Commission, which have not in the past been able to agree on a source of water for the North Jersey area, reached agreement at a meeting on July 10. At that time the boards resolved that, after conferences with interested local officials, proposals would be made to New York and Pennsylvania state officials leading to a joint tri-state development of the Delaware River watershed.

It is stated that at present the North Jersey area, with a population of more than 3,000,000, has developed water resources capable of supplying only 350 m.g.d. While this is sufficient for present needs, shortage is feared in the near future.

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Committee to Study Silicosis

The New York State Industrial Commissioner has appointed an advisory committee on the control of silica dust in rock drilling. The committee will make a comprehensive study leading to recommendations concerning dust collecting devices to be employed in hazardous industries as required by a recent state law (*ENR*, June 11, 1936, p. 862).

The committee will be made up of representatives of the medical profession, the state labor department, the state federation of labor, contractors engaged in rock drilling, and a number of specialists on rock drilling and dust control equipment.

◆

Brief News

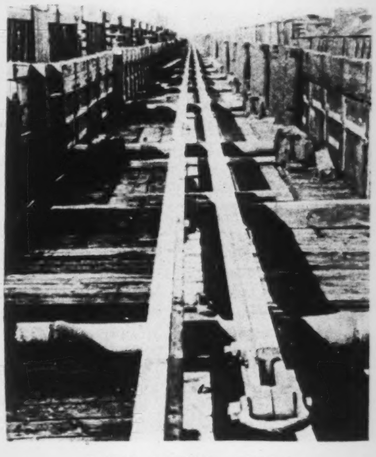
The Louisiana Senate has defeated a measure (*ENR*, June 4, 1936, p. 825) which would have placed all engineers and surveyors under the control of the state board of engineering examiners. The measure had previously passed the lower branch of the legislature.

PWA Has Advertised for bids, to be opened Sept. 1, on the superstructure of Andrew Jackson Court, a slum clearance project in Nashville, Tenn. The first bids which were received on this project were thrown out as excessive (*ENR*, July 23, p. 134).

Intended to Forward PWA Activities in Louisiana, an act signed by the governor authorizes suspension of an act of 1935 which requires the approval of the state bond and tax board before loans or grants from the federal government can be accepted by parishes or cities.

Contract for Repair and completion of the irrigation canal and distributing system of the Maverick County Water Control and Improvement District has been let to the Hardwick Co., Inc. The district has already spent about $7,000,000 on the project, which supplies irrigation water from the Rio Grande River. Work on the new contract is expected to start at once.

The Supplementary Estimates introduced into the federal House of Commons just before its adjournment included an item of $500,000 for improvement works on the Richelieu River. This vote has attracted attention in view of current discussions reviving the idea of a St. Lawrence to Hudson River seaway, via the Richelieu River.



RAILS 4,000-FT. LONG

Rails welded into two units, each 4,000-ft. long, were laid recently in the Blossburg tunnel of the Northern Pacific Ry., near Helena, Mont. The rails were laid out in the bottom of 90 gondola cars for welding and delivery to the tunnel. Box-car axles, supported in journal boxes bolted to the floor, provided rollers over which the rails moved as they were drawn from the cars in the tunnel. A similar operation was recently completed in the Bozeman tunnel of the same railroad. (*ENR*, July 16, p. 100).

The Calcutta Port Commissioners have awarded the contract for the new Howrah bridge over the Hooghly River to the Cleveland Bridge & Engineering Co., of London, England, on a bid of $8,000,000. The lowest bid, that of Krupp, of Germany, was ruled out on the ground that the contract might prove impossible to carry out because of the disturbed political condition in Europe. With a main span of 1,500 ft., the new bridge will be one of the largest in the world. Among cantilever bridges it will rank next in size to the Forth bridge (three spans of 1,700 ft.), and the Quebec bridge (1,800 ft.).

The Connecticut Board of Finance and Control has authorized the State Board of Civil Engineers to proceed with a survey of dams. It is proposed to obtain useful data on old dams throughout the state and to determine whether, and to what extent, the March floods have weakened them. A special appropriation of $2,000 has been added to the usual small appropriation to cover this work.

Ohio Will Spend an estimated $8,804,753 on relocating 150 miles of highway affected by the Muskingum Valley Conservancy District flood control and water conservation project. This amount will be spent in the fourteen reservoir areas of the district over a three-year period, with the first construction year now nearly completed. A total of 36.7 miles of highway has already been completed or is under construction, at a cost of $1,874,978.

Personals

W. A. Thomas has resigned as city engineer of Ferguson, Mo., to take a position with the A. D. Gates Construction Co. of St. Louis.

Jules Duchastel de Montrouge, general manager since 1924 of the Quebec Forest Industries Association, has been appointed assistant general manager for the Port of Montreal.

T. L. Tremblay has resigned his position as general manager of the Port of Quebec, but is being retained as port adviser. He is succeeded by **Louis Beaudry**, at present chief engineer of the port.

James H. Cissell has resigned from his position as engineer of bridge design for the Michigan State Highway Department to resume his duties as professor of structural engineering at the University of Michigan, where he has been a member of the faculty since 1915.

S. J. Fette, who has been on leave from his position as associate professor of sanitary engineering at the Virginia Polytechnic Institute to act as associate director of the finance and control division of the Resettlement Administration, has returned to his duties at the Polytechnic Institute.

Paul D. Hodges, associate engineer for the U. S. Indian Service, previously engaged in designing and supervising construction of dams on Indian reservations in North Dakota, has been assigned to investigate and study surface and groundwater supplies for the San Carlos irrigation project at Coolidge, Ariz.

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Obituary

V. H. Sikes, 61, assistant engineer of structures for the New York Central Railroad, died on August 1.

C. Cecil Travis, a civil engineer of Los Angeles, died there recently at the age of 60.

Forbes L. McKenzie, a Boston contractor, died in Belmont, Mass., on July 30 at the age of 74.

James Kay, an architect of New York City, died in Nashville, Tenn., on July 26, at the age of 57.

James D. Saunders, a civil engineer and formerly city engineer of Indianapolis, Ind., died in Indianapolis at the age of 82, on July 21.

Clinton Llewellyn Cole, a civil engineer connected with the H. Wales Lines Construction Co. of Meriden, Conn., died in West Hartford, Conn., at the age of 57 on July 29.

Sterling G. Thomas, a construction engineer on the staff of the Stone & Webster Engineering Corp., died in Newton, Mass., on July 27 at the age of 36.

Robert McDonough, 30, a civil engineer engaged in relocation of railroad track near the Tygart Valley dam in West Virginia, was killed on July 29 when he was struck on the head by falling timber in a tunnel.

M. J. O'CONNOR, a retired architect of New York City, died July 25 in Mt. Kisco, N. Y. Prior to his retirement Mr. O'Connor was a partner in the firm of Little & O'Connor.

W. VERNE MCCOWN, 48, an engineer for the United States Bureau of Public Roads, died recently. Mr. McCown had been engaged in various capacities in highway construction work, since his graduation from Iowa State College in 1912.

WALTER H. HOFFMAN, formerly secretary of the Louisiana Board of State Engineers, died in New Orleans on July 28. Mr. Hoffman was a graduate of Washington and Lee University and was for many years a civilian engineer for the Corps of Engineers, U. S. Army.

JAMES H. CURTIN, a resident engineer for the Triborough Bridge Authority, died in New York City on July 1 at the age of 52. Mr. Curtin was a graduate in civil engineering of the University of Wisconsin. He served as a field engineer on many large bridge projects, including the Hell Gate bridge, the Niagara River arch bridge, the Kill van Kull bridge, the Pulaski Skyway, the Triborough bridge.

CONSTRUCTION STATISTICS FOR THE WEEK

ENGINEERING construction awards for the week exceed $50,000,000 for the eighth time this year with a total for this week of $53,253,000 compared with the average week to date this year of $43,788,000 and the corresponding week's volume a year ago of $35,596,000. This week's total is made up of private construction, $14,678,000; and public, $38,575,000; of which federal is $15,138,000 and state and municipal, $23,437,000. Corresponding values a year ago are: private, $9,298,000; public, $26,298,000; federal $13,443,000; state and municipal, $12,855,000.

Heavy awards for highways, the largest commercial building volume for any week in more than three years, and high awards for earthwork and waterways account for the increased total this week. The subtotals for the various classes of work for this week are: highways, $12,665,000; earthwork, drainage, $9,516,000; commercial buildings, $9,736,000; industrial buildings, $4,301,000; public buildings, $9,125,000; bridges, $2,654,000; sewerage, $1,445,000; waterworks, $2,158,000; unclassified, $1,653,000.

The larger awards include: malt house, Milwaukee, Wis., $400,000; phosphate mining plant, Monsanto Chemical Co., Monsanto, Tenn., $500,000; group residences, Stony Point Land Co., Woodlawn, N. Y., $500,000; college unit, Stanford University, Palo Alto, Calif., $525,000; administration building and other buildings at Missouri State Penitentiary, Jefferson City, Mo., $2,003,000; superstructure Meeting Street Manor and Cooper River Housing Project, Columbia, S. C., $973,000; U. S. Narcotic Farm Buildings, Fort Worth, Tex., $1,068,000; highways, Mississippi, $1,378,000; Michigan, $1,157,000; Missouri, $1,575,000; West Virginia, $1,060,000; Tennessee, $693,000; state roads in Lynnfield and Peabody, Mass., $860,000; bridges, Missouri, $865,000; waterworks tunnel from Edisto River to Goose Creek, day labor, Charleston, S. C., $1,000,000; Conduit 16, Moffat Tunnel Extension Project, Denver, Colo., $466,000; dredging barge canal from Lock 2 to Lock 5, New York, $1,919,000; dredging Sections 3A and 4A, Calumet Harbor, Illinois and Indiana, $512,000; spillways at Horse Mesa Dam, Salt River project, Arizona, $713,000; Dam 21, Mississippi River, Quincy, Ill., $2,850,000; Dam 9, Mississippi River, Lynxville, Wis., $1,878,000.

New capital includes state and municipal bond sales, $2,288,000, and corporate security issues, $1,850,000, plus the first group of PWA allotments under the 1936-37 $300,000,000 fund. These total $24,043,000, making total new capital this week, $28,181,000.

CONTRACTS

(Thousands of Dollars)

	Weekly August 1935	Average Prev. 4 Weeks	Week Aug. 6 1936
Federal Government	$12,611	$11,814	$15,138
State and Municipal	12,396	20,248	23,437
Total public	$25,007	$32,062	$38,575
Total private	6,439	15,220	14,678
Week's total	$31,446	$47,282	$53,253

Cumulative to date:
1935...$800,794,000 1936..$1,401,223,000

NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

1936	Week Aug. 6	Cumulative
State and municipal	$2,288	$312,404
PWA non-federal	24,043	56,233
RFC loans	....	12,479
Corporate issues	1,850	150,987
PWA loans, Private	....	*—1,665
Total Non-Federal	$28,181	$530,438
Federal	....	691,237
Total new capital	$28,181	$1,221,675

Cumulative to date:
1935..$1,267,428,000 1936..$1,221,675,000

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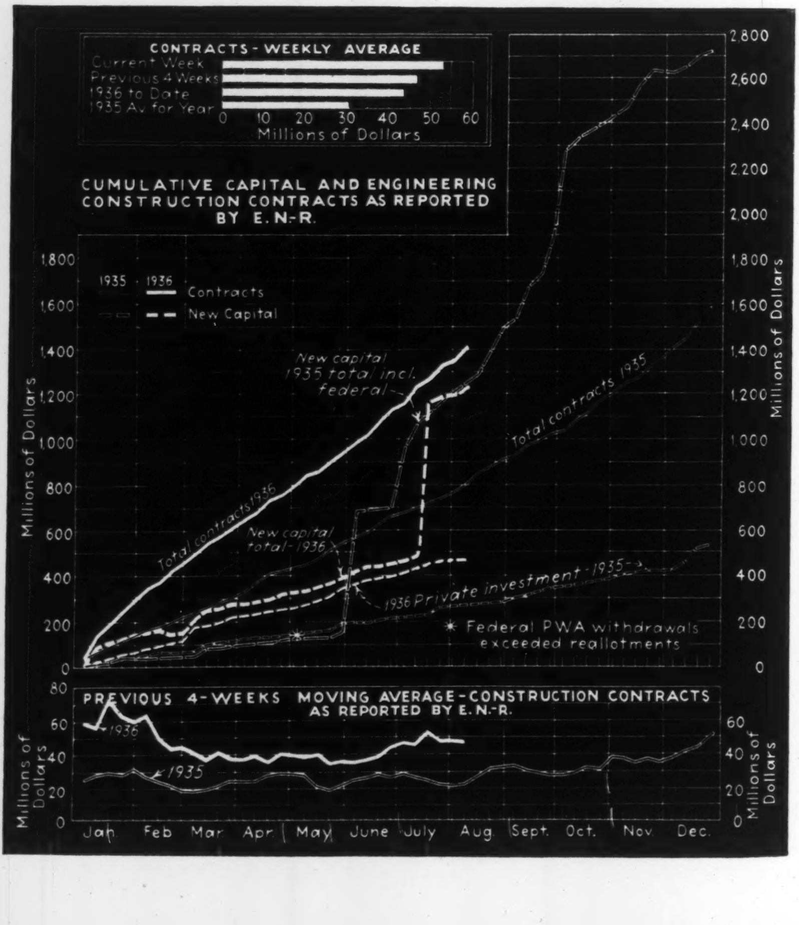
*Bond sales in this classification exceed reallotments during current year.

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities.

INDEX NUMBER

ENR Cost	1913 =100	1926 =100	ENR Volume	1913 =100	1926 =100
Aug., 1936	**208.10**	**100.03**	**July, 1936**	**188**	**82**
July, 1936	204.40	98.25	June, 1936	194	85
Aug., 1935	195.10	93.82	July, 1935	96	42
1935 (Av.)	195.22	93.84	1935 (Av.)	135	58
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47

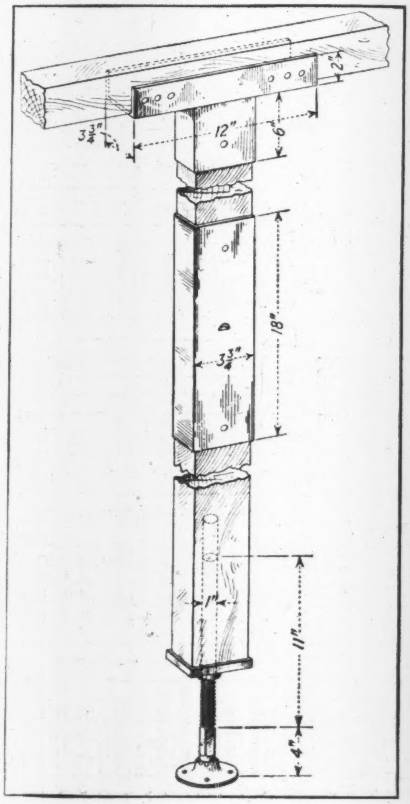


Construction Equipment and Materials

Low-Sensitivity Controller

A new line of controllers for temperature, time-temperature, flow, liquid level, pressure, time-pressure, and humidity, intended primarily for applications where a greatly reduced sensitivity is required and the load conditions fluctuate over a wide range, has been developed by the Bristol Company. This new unit, the Master Three-Vane Controller, is a pneumatic-type controller, equipped with a sensitivity adjustment, an automatic reset, and accessory features, which are said to minimize hunting, prevent departure from the control setting, make it possible to reach a new control setting without over-shooting, and make it possible to adjust the anticipatory effect to fit the process lag. This controller is intended to take care of the more severe conditions encountered in processes, such as sudden load changes and over-shooting.

SHORING EQUIPMENT developed by Construction Specialties Co., of Pittsburgh consists of three units: a jack with a 10-in. adjustable range, which supports a 4x4 timber; a splicing unit to join lengths of 4x4 timber; and a Tee head, fitting over the last 4x4, which takes bearing on the studs of the roof to be supported. The splice and Tee head units are of 14 gage annealed steel, welded. The 10 in. jacking range is said to make possible easy adjustment for settling of mud sills.



New Equipment in Brief

Crusher—A line of small portable crushing plants is now being brought out by the Austin-Western Road Machinery Co., of Aurora, Ill. Each unit consists of a crusher, a folding elevator and a motor, all mounted on a steel-tired truck. The units are powered by a Waukesha motor with belt drive. Eight sizes are available with jaw openings ranging from 9"x16" to 18"x38".

Signal—A flashing Neon signal that is intended to be used by contractors to warn of road construction has been developed by the Blatchford Corporation of Chicago. The unit is powered by a 1½ volt dry cell battery and requires no external wiring. The battery is said to be capable of operating the sign 24 hrs. a day for 120 to 230 days.

Hoist—Conco hand-chain hoists, manufactured by the Conco Engineering Works of Mendota, Ill., now are provided with precision ball bearings.

Welding Rod—The Hobart Bros. Co. of Troy, Ohio, recently introduced "Thinweld," a specially coated electrode for arc welding light-gage sheet metal. It is claimed that metal as light as 24 and 26 gage has been welded with this rod.

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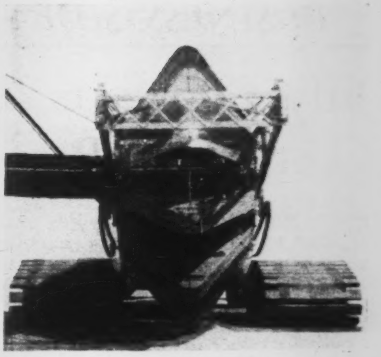
Portable Compressor

An 85 cu.ft. per min. compressor, manufactured by the Chicago Pneumatic Tool Co., is designed for extreme portability. It is a two-stage, air cooled, gasoline-engine driven unit. The weight of the unit is 1,850 lbs. With an overall length of only 71½ in., and a width of 30 in., it can be mounted crosswise on a truck, taking up little room. The compressor is equipped with a lifting bale for easy handling with the crane. The design of the Utility Compressor closely follows that of other C. P. compressors.



NOT AN ALLIGATOR, this is a 50-ton wheel-type ditcher, recently manufactured for use in Venezuela by the Buckeye Traction Ditcher Co. The traction wheels, which are 6 ft. in width bring the bearing pressure down to about 500 lb. per sq.ft., enabling the machine to travel over swamp lands. The machine is 23 ft. wide, 17 ft. high, and 49 ft. long. It is powered by a 125 hp. diesel motor.

Truck-Mounted Loader

A new bucket-type loader mounted on a truck chassis is built by the George Haiss Manufacturing Co., of New York City. The revolving paddles of the automatic feeding device dig the material from a path wider than the truck and pass it into buckets by which it is elevated to loading height. Two discharge chutes are provided, one on either side. Power for the loading apparatus is provided by a 20 hp. gasoline engine mounted on the truck. Movement into the pile is accomplished by the reverse traction drive of the truck. A clean-up scraper, which works with the feeding propellers, cleans a smooth floor as the machine digs into the pile. The rated loading capacity is 1½ to 2 cu.yds. per min.

In moving from place to place, the elevator is swung to a horizontal position above the cab of the truck. This is done by turning a hand wheel. The height of the rig when in traveling position is only 10 ft. 8 in.

One man only is needed for loader operations. Ordinarily the truck driver operates the loader.

Recent Publications

- A description of the operating principles and mechanism of *Micromax Thermocouple Pyrometers,* together with a catalog of the various models, is given in a new 52-page pamphlet of that name, issued by the **Leeds & Northrup Company** of Philadelphia, Pa. • *The Art of Proportioning,* a mimeographed pamphlet issued by **D. W. Haering & Co.,** of Chicago, is a dissertation on proportioning solutions of liquids in liquids, using H-O-H mixers. • The **Abart Gear & Machine Company,** present, in their *Bulletin 1200,* machine drawings and tabular descriptions of their straight line drive spur-gear speed reducers. • **The Bonney Forge & Tool Works** of Allentown, Pa., has recently issued *Catalog No. 36,* a catalog of their tools.
- *Catalog L,* a handsomely lithographed 24-page booklet issued by Littleford Bros., of Cincinnati, Ohio, describes their road maintenance and construction equipment. • The **Carrier Engineering Corporation,** Newark, N. J., describes its Kroy-type unit heater, its application, dimensions, ratings, and specifications, in the 12-page *Bulletin No. 46-E-1* • The *Holophane catalog* in 43 pages describes lighting equipment of the **Holophane Company** of New York.
- Catalogs of *Soil Testing Instruments and Apparatus* are hard to come by. **The American Instrument Co.,** of Silver Springs, Md., has just issued one in 20 pages. • **E. F. Houghton & Co.,** of Philadelphia, have begun publication of *Research Illustrated.* The publication will present the results of their research on belting problems. • The 1936 edition of the **B. F. Goodrich Company's** (Akron, Ohio) *Operators Handbook* gives specifications on many types of tires.

BUSINESS SIDE OF CONSTRUCTION

July Contracts Double Year Ago Cost Index Rises Above 1926

ENGINEERING awards in July totaled $226,595,000 for 5 weeks, a gain of 113 per cent over a year ago, comparing weekly averages. Of this total, $65,382,000 is for private work and $161,213,000 for public. Of the public, $54,959,000 is federal. Corresponding values for 4 weeks for last July are: total, $86,873,000; private, $22,430,000; public, $64,443,000; federal, $17,862,000.

Geographically, all territories show strong gains over last year and New England, South, Middle West and Far West show gains over June. Middle Atlantic and West of Mississippi which hold the widest gains over last year of any territories show fewer awards in July than in June, but continue their average lead over last year.

Heavy awards for PWA housing superstructures swelled the public building volume in July beyond the high figure in June and exceeded the average week a year ago by 350 per cent. Industrial buildings gained slightly over June but more than doubled last July's volume. Commercial buildings though not as active as last month doubled the rate of awards a year ago. Highway awards dropped below June but doubled the rate of awards a year ago. Bridges continue a strong lead over last year.

New Capital

State and municipal bond sales totaled $35,861,000 and corporate securities issues, $30,163,000. Federal appropriations, which became available in July with the beginning of the new fiscal year, totaled $686,282,000.

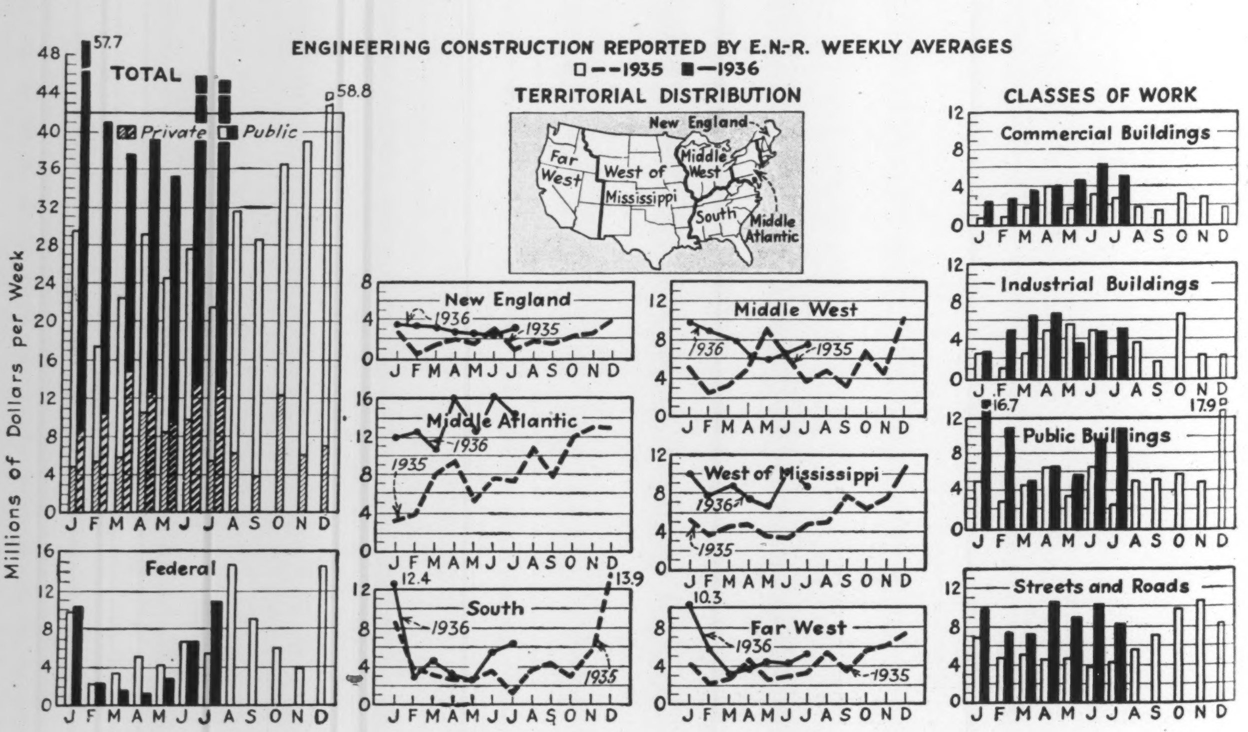
Materials

Cement—The Portland cement industry in June, 1936, produced 11,273,000 barrels, shipped 12,417,000 barrels from the mills, and had in stock at the end of the month 19,287,000 barrels. Production and shipments of Portland cement in June, 1936, showed increases of 29.2 and 62.7 per cent respectively, compared with June, 1935. Portland cement stocks at mills were 16.4 per cent lower than a year ago. The total production for the first half of 1936 amounts to 43,124,000 barrels, compared with 33,637,000 barrels in the same period of 1935, and the total shipments for the first half of 1936 amounts to 46,810,000 barrels, compared with 31,933,000 barrels in the same period of 1935 as reported by the Bureau of

ENGINEERING CONSTRUCTION CONTRACTS REPORTED IN JULY, 1936

Five Weeks—Thousands of Dollars (000 Omitted)

	New England	Middle Atlantic	South	Middle West	West of Mississippi	Far West	United States			Canada
							July 1936	Seven Months 1936	Seven Months 1935	July 1936
Public Works										
Waterworks	2,863	570	429	603	1,409	1,219	**7,093**	66,356	37,282	210
Sewerage	329	6,528	247	3,938	111	343	**11,496**	81,421	40,080	35
Bridges, public	536	6,780	1,331	2,502	8,354	1,389	**20,892**	106,046	39,757	1,308
Earthwork and waterways	484	4,186	2,067	2,012	6,710	4,907	**20,366**	97,823	106,444	113
Streets and roads	1,489	9,686	8,293	4,618	13,042	4,461	**41,589**	279,375	144,749	4,031
Buildings, public	5,982	14,834	12,077	6,594	9,266	3,223	**51,976**	291,112	132,537	481
Unclassified, public	501	1,630	1,376	808	1,646	1,840	**7,801**	61,825	31,998	190
Total public	12,184	44,214	25,820	21,075	40,538	17,382	**161,213**	983,958	532,847	6,358
Federal gov't (included in above classifications)	5,468	11,958	12,560	7,533	11,568	5,872	**54,959**	171,227	155,556	.....
Private										
Bridges, private	179	3,534	30	50	139	......	**3,932**	8,132	8,218	.....
Buildings, industrial	1,546	3,317	3,616	11,587	1,924	2,956	**24,946**	150,555	99,044	1,423
Buildings, commercial	1,367	16,155	951	3,610	1,073	1,866	**25,022**	127,498	62,412	871
Unclassified, private	227	4,154	1,027	1,066	537	4,471	**11,482**	77,827	41,623	23
Total private	3,319	27,160	5,624	16,313	3,673	9,293	**65,382**	364,012	211,297	2,317
July, 1936 (5 weeks)	**15,503**	**71,374**	**31,444**	**37,388**	**44,211**	**26,675**	**226,595**	......	......	**8,675**
June, 1936 (4 weeks)	11,800	64,433	23,167	26,483	41,261	16,773	**183,917**	......	......	11,092
July, 1935 (4 weeks)	3,921	29,918	4,999	14,437	20,900	12,698	**86,873**	......	......	5,431
Seven months, 1936	94,224	418,781	172,507	232,228	264,724	165,506	......	1,347,970	......	53,125
Seven months, 1935	50,296	191,827	114,985	152,342	132,978	101,716	......	......	744,144	39,182


ENGINEERING CONSTRUCTION REPORTED BY E.N.-R. WEEKLY AVERAGES

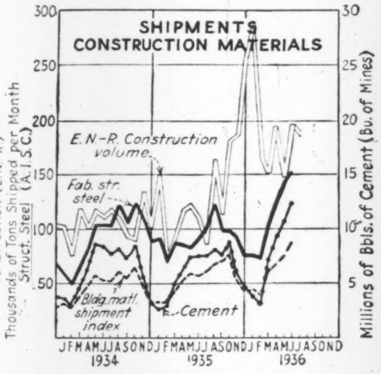
□--1935 ■—1936

Mines from all manufacturing plants except two.

The ratio of production to capacity for June, 1936, is 52.3 per cent compared with 39.6 per cent a year ago and 48.9 per cent last month.

Steel—The operating rate of steel companies having 98 per cent of the steel capacity of the industry and reporting to the American Institute of Steel Construction averaged slightly over 70 per cent for July, compared with 38.8 per cent for the same period last year.

Shipments of fabricated structural steel during June were the largest for any month since August, 1931, according to the final reports received by the American Institute of Steel Construction. The June bookings amounted to 54.9 per cent of normal (normal being the annual average for the years 1928-1931, inclusive). The bookings for the six months of the current year were better than the tonnage booked in any same six months period since 1931.



The shipments during June amounted to 64.4 per cent of normal. Despite the rush of shipments out during this month the total shipments for the six months period ending with June averaged but 48.2 per cent of normal.

Lumber—Lumber production ranged from 60 to 69 per cent of 1929 weekly average and shipments 60 to 64 per cent of 1929 weekly average according to reports of National Lumber Manufacturers Association. Production of hardwoods and softwoods combined, totaled 245,169,000 feet during the week ending July 18; shipments 222,808,000 feet and booked orders 221,431,000 feet.

Shipments Index—The index of shipments of construction materials, compiled by Advance Construction Programs Unit, Space Control, rose to 89 for June compared with 72 a year ago and 75 last month.

Building Permits

The building permit volume for 215 cities reached a total of $112,640,106 in June, according to the monthly summary prepared by Dun & Bradstreet, Inc. This exceeded all previous records since May, 1931, and revealed an increase of 36.5 per cent from the May figure of $82,510,596. Comparison with the June, 1935, total of $52,672,794 showed a rise of 113.9 per cent. The expansion in building operations was particularly marked in New York City, especially in the boroughs of Manhattan and Bronx. In the latter borough, there was a feverish rush to file plans in order to escape the more stringent requirements of the proposesd new city building code. The total for the five boroughs of New York City aggregated $40,011,404. This was the highest since April, 1931, and reflected a gain of 118.7 per cent over May and of 206.0 per cent over June a year ago.

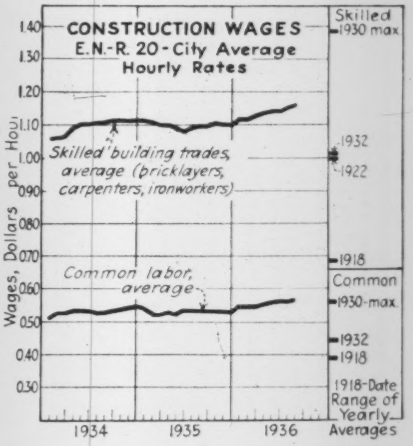
For the 214 cities outside of the Metropolis, the June permit total amounted to $72,628,702, against $64,217,914 in the preceding month and $39,596,123 in the corresponding month last year, giving increases of 13.1 and 83.4 per cent, respectively.

Labor

Employment in the production of six major construction materials as reported by Advance Construction Programs Unit, Space Control, is 54.5 for June compared with 52.5 in May and 44.6 in June, 1935.

Further gains in employment were shown in the private building construction industry in June, 1936, according to the U. S. Bureau of Labor Statistics. Reports received from 9,514 contractors employing 105,567 workers in June showed an increase of 4.4 per cent in employment over the month interval and 5.2 per cent in payrolls. Employment in this industry in June, 1936, was 21.5 per cent above the June, 1935, level and payrolls were 38.4 per cent higher. The information furnished by reporting contractors relates to employees engaged in erecting, altering and repairing private buildings and does not include projects financed by the Public Works Administration, Reconstruction Finance Corporation funds, or regular appropriations of the Federal, State and local governments.

During June approximately 3,290,000 workers were employed on the Works Program financed from funds provided by the Emergency Relief Appropriation Act of 1935. Compared with the previous month, this is an increase of 176,000 in the number of employees. Of the total number engaged on this program in June, 2,561,000 were working on projects operated by the Works Progress Administration.



Over 345,000 workers were engaged on the Public Works Administration construction program in June. This includes 170,000 employees working on projects financed from E.R.A.A. 1935 funds, who are also included under the Works Program. Compared with the previous month, employment on the Public Works Administration construction program in June showed an increase of 30,000 workers.

More than 102,000 workers were employed in June on construction projects financed by regular governmental appropriations. This is an increase of 25,000 compared with the previous month.

◆

ENR Cost and Volume Index

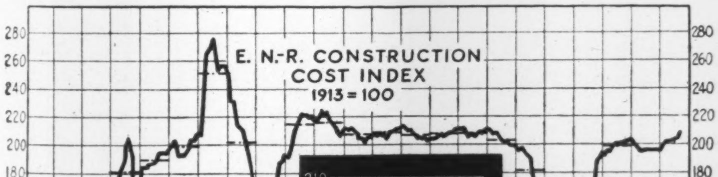
The *Engineering News-Record* Construction Cost Index jumps to 208.10 for August due to an increase in structural steel price and continued stiffening of common labor rates. This is the first time the 1926 average, 208.03 has been exceeded since the cost index has been climbing away from the 1932 low values.

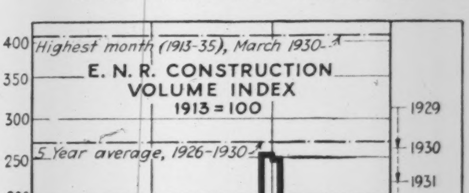
A correction is reported in the July cost index due to a correction in the common labor wage average. The corrected value for July is 204.40.

The *EN-R* Volume Index is 188 for July compared with 194, the corrected value for June. This correction is due to the change in value of the construction cost index.

INDEX NUMBER

ENR Cost	**1913 = 100**	**1926 = 100**	**ENR Volume**	**1913 = 100**	**1926 = 100**
Aug., 1936	**208.10**	**100.03**	**July, 1936**	**188**	**82**
July, 1936	204.40	98.25	June, 1936	194	85
Aug., 1935	195.10	93.82	July, 1935	96	42
1935 (Av.)	195.22	93.84	1935 (Av.)	135	58
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47





CURRENT PRICES OF CONSTRUCTION MATERIALS

CEMENT, AGGREGATES AND READY-MIXED CONCRETE—F.O.B. CITY

	PORTLAND CEMENT Per bbl., carload lots, including 40c. per bbl. for bags, cash discount not included			SAND AND GRAVEL Per ton, carload lots			CRUSHED STONE Per ton, carload lots		CRUSHED SLAG Per ton, carload lots, f.o.b. plant		CONCRETE Ready Mixed 1:2:4, 50c.y. or more, delivered
	Cloth Bags*	Paper Bags	Bulk	Gravel, 1½ in.	Gravel, ¾ in.	Sand	1½ in.	¾ in.	1½ in.	¾ in.	
Atlanta	$2.83	$2.58	$2.50	$1.60	$1.60	$1.55	$1.69	$1.92	$1.69	$1.69	$8.50
Baltimore	2.66	2.41	2.21	1.80‡	1.80‡	1.30‡	1.80‡	1.80‡	1.80	1.80	7.35
Birmingham	2.40	2.15	1.95	1.80	1.80	1.45	1.00	1.00	.90	1.15	5.45
Boston	2.42	2.12	1.92	1.40	1.40	.90	1.40	1.40	.....	.....	6.50
Chicago	2.60	2.35	2.20	1.50	1.50	1.80	1.32	1.32	1.00	1.00	.....
Cincinnati	2.50	2.25	2.05	1.20	1.20	1.30	1.80	1.80	.....	.....	6.70
Cleveland	2.41	2.16	.....	1.40	1.40	1.40	1.60	1.60	1.75	1.75	7.85
Dallas	2.45	2.20	2.00	1.55	1.55	1.30	1.95	2.05	.....	.....	7.00
Denver	3.09	2.84	.....	1.20	1.40	.90	.....	.....	.....	.....	.....
Detroit	2.25	2.00	1.80	1.25	1.25	1.10	1.50	1.75	1.35‡	1.45‡	5.95
Kansas City	2.51	2.26	2.06	1.85	1.85	1.25	1.85	1.85	.....	.....	7.65
Los Angeles	2.32	2.12	.....	1.20	1.20	.90	1.20	1.20	.....	.....	6.31
Minneapolis	2.90	.....	2.45	1.00p	1.00p	.25p	1.00p	1.00p	.....	.....	6.75d
Montreal	.....	2.62	1.70	.85	.90	1.00	.85‡	1.20‡	.....	.....	4.50
New Orleans	2.47	2.22	2.00	1.50	1.50	1.25	.....	.....	.....	.....	8.25
New York	2.70‡	2.45‡	.....	1.20†	1.20†	.70†k	1.75†t	1.85†k	.....	.....	8.00
Philadelphia	2.68	2.43	2.23	1.45‡	1.55‡	1.15‡	2.60‡	2.75‡	1.00‡	1.00‡	8.00D
Pittsburgh	2.46	2.21	1.61	1.50‡	1.50‡	1.70‡	2.25‡	2.25‡	1.50	1.50	7.00
St. Louis	2.53	2.28	.....	1.55‡	1.40‡	1.25/1.40‡	.75p	.75p	.65/.95§	.50/.95§	6.95
San Francisco	2.72	2.52	2.20	1.35	1.35	1.35	1.50	1.50	.....	.....	7.53
Seattle	2.95	2.70	.....	1.00	1.00	1.00	1.44	1.44	.....	.....	6.75

*10c. allowed for each returnable bag. †Per cu. yd. ‡Delivered. §F.o.b. Granite City, Ill. kBarge lots alongside dock. pf.o.b. plant. t5 tons or more. sWithin 3-mi. from Pub. Square. tton. d 5% discount for cash. D Discount 35c. 500 to 2000; 70c. 2000 to 5000; 95c. over 5000.

CURRENT MAXIMUM PRICES RECEIVED AT CEMENT MILL

Charge for bags not included. For cloth bags, add 40c. per bbl.; 10c. refund allowed for each returnable bag; for paper bags add 15c. per bbl., not refundable.

	Bagged	Bulk		Bagged	Bulk		Bagged	Bulk
Alpena, Mich	$1.50	$1.45	Ironton, Ohio	$1.50	$1.45	North Birmingham, Ala	$1.75	$1.70
Buffington, Ind	1.70	1.65	Iola, Kans	1.70	1.65	Richard City, Tenn	1.85	1.80
Crestmore, Calif	1.72	....	La Salle, Ill	1.70	1.65	Saginaw, Mich	1.40	1.35
Dallas, Tex. (Inc. 5c. tax)	1.80	1.75	Limedale, Ind	1.70	1.65	Steelton, Minn	1.80	1.75
Hannibal, Mo	1.70	1.65	Mason City, Ia	1.70	1.65	Universal, Pa	1.65	1.60
Hudson, N. Y	1.75	1.70	Norfolk, Va	1.76	1.71	Waco, Tex. (Plus 5c. Tax in Texas)	1.75	1.70
Independence, Kans	1.70	1.65	Northampton, Pa	1.65	1.60	Wyandotte, Mich	1.45	1.40

Cash Discounts, Cement, to Contractors: truck delivery, 2% for cash on 10th of month; on carload deliveries, no trucking, same as dealer cash discount, 10c. per bbl. for payment within 15 days of date of invoice.

STRUCTURAL CLAY BUILDING TILE, BRICK AND LIME—DELIVERED

	STRUCTURAL CLAY TILE—PARTITION—SCORED Per M, lots of 2,000 pieces or over,			STRUCTURAL CLAY TILE—LOAD BEARING—SCORED Per M, lots of 2,000 pieces or over,			BRICK Per M, in quantity,		LIME Per ton, in paper, carload lot		
	3x12x12 in.	4x12x12 in.	8x12x12 in.	8x12x12 in.	10x12x12 in.	12x12x12 in.	Common backing	Straight hard	Hydrated finishing	Common hydrated	Pulverize or lump
Atlanta	$71.10	$75.80	$142.10	$170.50	$199.00	$227.40	$12.25	$14.25	$18.50	$13.00	$13.00
Baltimore	91.00	98.00	183.00	219.00	315.00	390.00	13.00	17.00	20.00	14.00	25.00
Birmingham	78.75	84.00	157.50	189.00	201.25	242.00	15.00	19.00	3.00§	1.75§	2.25§e
Boston	79.90t	85.20t	159.75t	181.05t	237.30t	271.20,	14.50	18.00	20.00	15.50	3.50q
Chicago	66.00	70.40	132.00	174.30	214.40	245.00	10.20	11.20	18.00	13.40	14.00
Cincinnati	58.85	62.80	117.75	141.30	169.26	209.56	17.50	17.50	15.97	12.48	.....
Cleveland	45.00	48.00	90.00	108.00	138.50	158.50	13.75	13.75	16.00k	14.00k	2.65k
Dallas	72.00	75.00	149.00	177.00	217.00	269.00	13.50	26.00	19.00	16.00	1.65p
Denver	78.50	88.00	141.50	166.50	196.00	225.00	11.50	15.00	30.00k	22.00k	22.50k
Detroit	71.85	76.65	143.70	230.00	280.00	325.00	12.50	13.50	18.00	15.00	15.25
Kansas City	72.50	75.50	92.00*	140.50	177.75	207.25	13.00	22.00	18.90	12.40	10.35
Los Angeles	77.50	94.25	134.00*	188.00	232.50	311.50	11.50	14.00	18.70	.....	19.70
Minneapolis	80.15s	85.50s	145.40s	167.75s	175.25s	243.70s	13.10	15.10x	25.50	21.00	23.00
Montreal	84.00s	96.00s	180.00s	......	......	......	20.75	31.00	15.00	10.00	7.00
New Orleans	69.10	69.10	129.50	155.40	203.40	251.80	14.00	.....	18.40	11.25	.....
New York	78.40	83.60	125.40*	193.30†	238.80†	295.60†	12.50	.....	20.00	15.50	20.00
Philadelphia	86.00	90.00	150.00	200.00	240.00	280.00	18.00	20.00	16.35	11.25	10.75
Pittsburgh	58.40c	62.30c	116.85c	165.00c	217.00c	250.00c	16.50	25.00	17.20	14.50	2.22p
St. Louis	62.00	65.00	120.00	180.00	234.00	280.00	15.00	18.00	22.00	16.00	2.25p
San Francisco	84.00	94.50	225.00	......	......	......	15.00	18.00	.55§	.40§	2.05p
Seattle	84.00	95.00	180.00	198.00	......	......	15.50d	18.50	30.00	20.00	28.60p

*6x12x12 in. †F.o.b. Perth Amboy, N. J. ‡Per sack. §Per bbl., 200 lbs. pPer bbl. 180 lbs. qper bbl. 280-lb. kLCL. dless $1 cash 15 days. elump. sSmooth. xSelected common. tCarload lots delivered to job.

ROAD SURFACING MATERIALS—F.O.B. CITY

	PAVING BRICK AND BLOCKS			PAVING ASPHALT Per ton, less than 80 penetration,		ASPHALT BINDERS—FLUXES Per gal., 80-300 penetration,		CUTBACK ASPHALT Per ton,		ROAD OILS Per gal.,	ASPHALT EMULSION Per gal.,	
	Granite per M, lots of 50,000, 4x4x8 in.,	Brick per M, 3x4x8½ in., carload lots	Wood per sq.yd., 3½ in., 16-lb. treat,	Tank car	Drums	Tank car	Drums	Tank car	Drums	Tank car	Tank car	Drum
Atlanta	$75.00	$35.00	$2.25	$17.40[2]	$2.62[2]	$0.0687[2]	$0.100[2]	$18.15[2]	$26.20[2]	$0.687[2]	$0.0875	$0.0875
Baltimore	108.00	44.00	2.25	17.00	22.00	.075	.10	.09†	.125†	.....	.....	.....
Birmingham	110.00	25.00	....	19.00	24.00	.0878	.1128	.10†	.137†	.....	.06	.....
Boston	75.00	26.00	2.75	17.00	22.00	.07e	.095e	.09†e	.12†e	.07e	.095e	.125e
Chicago	125.00	42.00	1.985	16.00	20.00	15.00	19.00	.07	.11	.045	.09	.135
Cincinnati	115.00	37.00	....	13.47	17.47	.12	.....	17.00	.....	.055	.08	.....
Cleveland	100.00‡	32.75	2.85	17.00	21.00	.07	.08	.075	.045†	.0525	.085	.095
Dallas	.....	30.00	....	18.00	21.75	.065	.16	18.75	26.25	.055	.085	.11/.135
Detroit	.....	37.00	....	16.80[4]	21.00[4]	.....	.....	15.25	20.25	.055	.11	.15
Kansas City	.....	40.50	....	18.50	23.50	17.50#	22.50#	.077†	.107†	.057	.11	.15
Los Angeles	.....	50.00	....	10.34	14.50	10.34#	14.50#	10.34	.....	.042	.06	.12
Minneapolis	.....	.....	1.75l	18.20	24.70	.098	.098	21.85	34.65	.048	.....	.....
Montreal	70.00	60.00	3.62	14.00	19.66	.07	.093	19.00	26.50	.07	.135	.165
New Orleans	100.00	32.00	....	15.20	18.75	.068	.093	.075†	.10†	.....	.0785	.11
New York	120.00§	65.00	2.75	17.00	22.00	.075	.10	.09†	.125†	.....	.075	.11
Philadelphia	110.00	45.00	2.25	13.50	18.00	.05	.08	.05†	.08†	.045	.08	.....
Pittsburgh	115.00	40.00	....	19.00	24.00	.0775	.1125	.086†	.121†	.0675	.....	.....
St. Louis	115/120	35.00	....	19.70	24.70	.08775	.1075	.0825†	.1125†	.045	.09	.12
San Francisco	.....	50.00	....	12.00	18.00	12.00#	18.00#	12.50#	23.00#	.04	.0575	.117
Seattle	.....	35.00	....	25.50	25.50	17.00#	23.00#	15.00	.....	.03	.19	.20

Note: Paving asphalt, tank car or boat, f.o.b. Maurer, N. J., per ton, Bermudez, $25.00; Trinidad, $21.00. *Per sq. yd. †Per gallon. ‡4½x6x5½in. #per ton **4x5x9-in. Av. [4]Mexican. [2]Local reduction due to 20% reduction intra-state class freight rates, only Georgia affected l 2½ in. 6 to 8-lb. treatment. e F.o.b. Everett

CURRENT PRICES OF CONSTRUCTION MATERIALS

IRON AND STEEL PRODUCTS—BASE MILL PRICES

	STRUCT. SHAPES-PLATE	REINF. BARS ¾-in. billet,	RIVETS ¾-in. structural	WIRE NAILS Base	SHEET PILING Base	STEEL RAILS Per Gross Ton Standard	Light	Re-rolled	TRACK SUPPLIES Angle bars	Std. spikes	Tie plates	Track bolts
Birmingham	$2.05	$2.10	$3.00	$2.55	.....	$36.375	$35.00	$34.00	$2.55	$2.60	$1.90	$3.60
Chicago	1.95	2.10	3.00	.....	$2.25	36.375	35.00	34.00	2.55	2.60	1.90	3.60
Pittsburgh	1.90	2.05	2.90	2.10	2.15	36.375	35.00	34.00	2.55	2.60	1.90	3.60

IRON AND STEEL PRODUCTS—F.O.B. WAREHOUSE, PER 100 LB., BASE PRICE

F.O.B. WAREHOUSE	STRUCTURAL SHAPES Per 100 lb., base price	REINFORCING BARS Per 100 lb., ¾ in., base price New billet	Rail steel	EXPANDED METAL LATH —Per 100 sq.yd., carload lots— Std. diamond mesh, 3.4 lb.	Std. ribbed 3.4 lb.	WELDED FABRIC REINFORCING —Per 100 s.f., carload lots— 4x16 in., No. 5 & 10 wires	4x12 in., No. 8 & 12 wires	6x6 in., No 6 & 6 wires Per sq.yd.	SHEET PILING Per 100 lb. base price
Atlanta	$3.36	$2.34	$2.19	$21.00	$23.00	$1.53	$1.16	$0.1449	$2.70
Baltimore	3.00	2.75	2.60	24.00	26.00	1.41	1.08	.1332	2.45
Boston	3.64	2.923	1.962	22.00†	24.00†	1.46	1.12	.1449	2.57
Chicago	3.20	2.10	1.95	22.00	24.00	1.40	1.08	.1323	2.35
Cincinnati	3.40	3.25	3.25	25.01	29.50	1.39	1.07	.1287	.....
Cleveland	3.31	2.75	2.60	20.00	22.00	1.37	1.06	.1287	.....
Dallas	3.45	2.825	2.675	22.00	28.00	1.63	1.23	.1548	3.005
Denver	3.95	3.76	3.61	22.00	25.00	1.68	1.26	.1584	....
Detroit	3.52	2.20	2.05	20.00†	22.00†	1.41	1.08	.1323	2.53
Kansas City	3.85	2.75	2.60	19.00	21.00	1.50	1.14	.1422	2.79
Los Angeles	2.35	2.475	....	23.00	27.50	1.68	1.26	.1584	2.65
Minneapolis	3.55	2.86	2.71	22.00	26.00	1.51	1.15	.1431	2.67§
Montreal	3.25	2.40	2.30	32.00	33.00	2.30	1.71	.238	....
New Orleans	....	....	....	20.50	23.50	1.54	1.17	.1458	....
New York	3.47	2.50	2.35	19.50	21.50	1.44	1.11	.1359	2.50
Philadelphia	3.10	3.06	2.22	20.00	24.00	1.42	1.09	.1332	2.56
Pittsburgh	1.90‡	1.85‡	1.90‡	20.00‡	22.00‡	1.34	1.04	.1260	2.25
St. Louis	3.55	2.45	2.30	20.00	23.00	1.42	1.09	.1341	2.30
San Francisco	3.15	2.475	2.325	23.00	28.00	1.68	1.26	.1584	2.60
Seattle	3.55	2.50	....	24.50	26.50	1.68	1.26	.1584	2.60

‡Base Mill Price. Note: Shapes, bars and piling subject to quantity discounts. §Mill Price plus freight to Minneapolis. †f.o.b. delivered.

PAINTING AND ROOFING SUPPLIES—F.O.B.

	RED LEAD Per 100 lb. in 500-lb. lots Dry	WHITE LEAD Per 100 lb. in 500-lb. lots# in oil	READY-MIXED PAINT Per gal., drums Graphite*	Aluminum†	Ferric Oxide‡	ROOFING SUPPLIES Carload lots, f.o.b. factory Rolls, slate surfaced, 85-90 lb., per sq.	Asphalt felt, per 100 lb.	Tar felt, per 100 lb.	Asphalt coating, per gal.	Tar pitch, 350-lb. bbl., per ton
Atlanta	$7.75	$11.00	$1.60	$2.50	$1.40	$2.02	$1.81	$1.81	$.36	$25.60
Baltimore	7.00	10.50	1.45	2.25	1.70	2.03	3.00	3.00	.35	25.00
Birmingham	7.25	11.00	2.35	2.35	1.85	2.01	1.77	1.77	.36	22.00
Boston	7.00	10.50	1.85	2.70	.05x	1.75	3.15	3.15	.43	20.00
Chicago	7.00	10.50	1.35	....	....	2.43	2.24	2.24	.32	33.00
Cincinnati	7.00	10.50	1.45	2.50	1.40	2.40	2.39K	2.39K	.36	21.00
Cleveland	7.00	10.50	1.70	2.45	1.40	1.55	1.75K	1.75K	.23	19.00
Dallas	7.25	11.50	1.80	2.10	....	2.35K	3.80	3.80	.14	35.00
Denver	7.50	11.25	2.55	....	....	2.25	3.55	3.55	.35	30.00
Detroit	7.00	10.50	1.65	2.30	1.40	2.55	2.80	2.55	.35	30.00
Kansas City	7.00	10.62½	1.65	2.65	1.10	1.78	3.10–3.20	3.10–3.20	.25	24.50
Los Angeles	7.25	10.75	1.60	2.25	2.30	2.00	1.50	....	.45	.....
Minneapolis	7.00	10.62½	....	....	....	1.73v	1.67Kv	1.67Kv	.29	23.80
Montreal	....	8.20	1.85	3.50	1.85	1.90	2.44	1.88	.55	1.52§
New York	7.00	10.50	1.75	2.35	1.40	1.60	1.52K	1.52K	.232	22.00
Philadelphia	7.00	10.50	2.50	2.95	1.50	1.68	1.86K	1.48K	.37	22.00
Pittsburgh	7.00	10.50	1.05	1.95	.80	1.75	1.65K	1.65	.38	25.00
St. Louis	10.00	10.50	1.85	2.45	1.30	1.83K	2.57	2.57	.25	25.00
San Francisco	7.25	10.75	1.80	2.75	1.10–2.00	1.89	3.10	3.10	.38	26.00
Seattle	7.25	10.75	1.80	2.85	2.20	3.30	2.66	2.60	.60	30.00

*U. S. War Dept. Spec. 3-49A. †ASTM Spec. D266-31. ‡80% maximum ferric oxide. §Per 100 lb. KPer roll, 65 lb. xPer lb. v Minneapolis & vicinity
Note: Red lead in oil 50c higher than white lead in oil. #Basis of quoting changed this month from list to 500-lb. lots.

MISCELLANEOUS ITEMS

	WINDOW GLASS Discounts from latest jobbers List, Sept. 15, 1928 Single or Double Thickness A quality	B quality	EXPLOSIVES Per lb. dynamite delivered 50-lb. cases Gelatin 40%	60%
Atlanta	85%	85%	$0.135	$0.15
Baltimore	88%	90%	.145	.16
Birmingham	88%	90%	.095	.11
Boston	88–10–5%	90–10–5%	.13	.145
Chicago	88%	89%	.135	.15
Cincinnati	89–10%	91–10%	.27	.29
Cleveland	85%	85%	.215	.225
Dallas	87%	89%	.1350	.1500
Denver	88%	89%	.145	.16
Detroit	87–10%	89–10%	.204	.235
Kansas City	85–10%	86–10%	.35	.15
Los Angeles	90%*	90–40%	.1725†	.1925†
Minneapolis	87%	88%	.145	.16
Montreal	60–30%	70–5%	.1825	.1950
New Orleans	88%	90%	.215	.23
New York	89%	90%	.215	.23
Philadelphia	89%	90%	.13	.1450
Pittsburgh	88%	88%	.10	.115
St. Louis	89%	89%	.13	.145
San Francisco	80%	85%	.1475	.1625
Seattle	90–10%	90–20%	.1425	.1575

*Disc. from list Aug. 1, 1929. †F.o.b. Arsenal. ‡Double thickness A & B.

CHEMICALS

Water, sewage treatment, road work, f.o.b. carlots, New York.

Bleaching powder, in drums, F.o.b. works, per 100 lb.	$2.00–2.10
Calcium chloride, 77-80%, flaked, in 400-lb. drums or 100-lb. moisture proof bags, f.o.b. works, freight equalized with points of competition, per ton	22.00–34.00
Chlorine cylinders, per lb. delivered	.055–.06
Silicate of soda, 52 deg., in drums, f.o.b. works, per 100 lb.	1.35
Soda ash, 58%, in paper bags, per 100 lb. dense	1.25
Sulphate of aluminum, commercial, in 100 lb. bags per ton	27.00–30.00
Sulphate of copper, in bbl., per 100 lb.	4.00–4.25

PILES

Prices per linear foot, pine, with bark on, f.o.b. New York; delivered from barge 1½ to 2c. per ft. additional:

Dimensions	Points	Length	Short Leaf Barge	Short Leaf Rail	Long Leaf Barge	Long Leaf Rail
12 in. at butt	6 in.	30 to 50 ft.	$0.12	$0.165	$0.13	$0.16
12 in.—2 ft. from butt.	6 in.	50 to 59 ft.	.16	.21	.15	.20
12 in.—2 ft. from butt.	6 in.	60 to 69 ft.	.17	.225	.16	.21½
14 in.—2 ft. from butt.	6 in.	50 to 69 ft.	.195	.235	.20½	.24½
14 in.—2 ft. from butt.	6 in.	70 to 79 ft.	.2125	.245	.24½	.30½
14 in.—2 ft. from butt.	5 in.	80 to 85 ft.	.275	.385	.....	.....
14 in.—2 ft. from butt.	5 in.	85 to 89 ft.	.315	.452	.....	.....

RAILWAY TIES

Prices f.o.b., per tie, for carload lots:

		6 In. x 8 In. by 8 Ft.	7 In. x 9 In. by 8½ Ft.
Boston	Oak, untreated	$1.10	$1.60
	Pine, untreated	1.25	1.70
	Pine, creosoted	1.65	2.30
New York	Long-leaf sap pine, untreated	.80	1.00
	Mixed oak, untreated	1.00	1.20
Birmingham	White oak, untreated	.85	1.35
	Red oak, creosoted	1.50	1.70
	Southern pine, untreated *6x8x8'6'	.55*	.70
	Southern pine, creosoted	1.10*	1.30
Chicago	White oak, untreated	1.15	1.40
	Oak, empty cell, creosoted	1.50	1.88
	Oak, zinc treated	1.40	1.70
	Southern pine, creosoted	1.35	1.78
Los Angeles	Fir, untreated	.50	.72
	Fir, creosoted	1.67	2.40
Philadelphia	Red oak, untreated	1.00	.....
	Red oak, creosoted	....	2.10
St. Louis	White oak, untreated	1.03	1.40
	Red oak, untreated	1.10	1.35
	Red oak, creosoted	1.45	1.95
	Sap Pine or cypress, untreated	.85	1.20
San Francisco	Douglas fir, green, untreated	.45	.63
	Douglas fir, empty cell, creosoted	1.14	1.56
Montreal	Birch or maple, untreated	1.00	1.10
	Birch or maple, creosoted	1.25	1.55

CURRENT PRICES OF CONSTRUCTION MATERIALS

PLUMBING, HEATING, WATER, SEWER AND DRAIN PIPE

	C. I. PIPE	**VITRIFIED SEWER PIPE**				**CLAY DRAIN TILE**		**CONCRETE SEWER PIPE**		**WROUGHT STEEL PIPE** Full standard weight. #			
	Per net ton, f.o.b. 6 in. to 24 in.*	Per foot, delivered, ASTM C 13-35				Per 1,000 ft., carload lots, f.o.b.		Per ft., delivered; ASTM C 14-35		1 to 3 in., Butt Weld		3½ to 6 in., Lap Weld	
		8 in., s.s.	12 in., s.s.	24 in., d.s.	36 in., d.s.	6 in.	8 in.	12 in.	24 in.	Black %	Galv. %	Black %	Galv. %
Atlanta	$43.00	$0.225dl	$0.405dl	$1.4625l	$4.50l	$80.00	$128.00	$0.45	$1.625	49.5	40.5	47	37.5
Baltimore	46.50	.28	.54	2.38	7.18	110.00	170.00	.40	1.60	....	....	....	....
Birmingham	39.00	.25	.45	1.625	4.66	85.00	220.00	.40	1.60	69.5	61.5	67	58.5
Boston	46.50	.224	.432	1.98	5.845	99.00	153.00	.42	2.35	....	....	....	....
Chicago	47.50	.245	.4725	2.025	4.60	60.00	120.00	.36	1.42	58.75	48.75	55.625	45
Cincinnati	46.00	.189	.3645	1.62	5.535	69.00	108.00	.40	1.40	58.75	48.75	55.625	45.00
Cleveland	47.50	.168	.324	1.44	4.1225	42.50	88.50	.32	1.58	61	51	57.9	47.25
Dallas	51.00	.35	.67	2.70	.....	165.00	260.00	.46	1.57	69.5	61.5	67	58.5
Denver	59.00	.27‡	.47‡	1.98‡	.....	105.00	180.00	.....	.....	50	40	47	36
Detroit	47.50	.203	.3915	1.71	5.2375	82.00	136.00	.35	1.20	59.3	49.3	56.1	45.5
Kansas City	49.00	.30	.54	2.16	5.00	120.00	140.00	.40	1.50	55	45	51.5	41
Los Angeles	47.50	.2825‡	.508‡	2.03‡	5.535‡	115.50	192.50	1.00	2.00	62.1	54.1	59.6	51.1
Minneapolis	49.50	.24‡	.432‡	1.728‡	4.60‡	.....	......	.65	1.65	....	....	....	....
Montreal	50.00	.33t‡	.59t‡	2.04t‡	.....	82.00	134.00	.51	2.04	....	....	....	....
New Orleans	45.00	.23	.41	1.60	....	67.00	......	.47	1.67	....	....	....	....
New York	46.00	.28	.55	2.35	6.35	.....	......	.55	2.00	57.50a	47.50a	54.375a	43.75a
Philadelphia	43.00	.336	.648	2.835	8.68	175.00	210.00	.54	1.80	52.37a	41.35a	53.39a	42.57a
Pittsburgh	49.00	.182‡	.351‡	1.62‡	4.92‡	69.00	108.00	.75c	1.75	69.5	61.5	67	58.5
St. Louis	46.00	.258	.459	1.83	3.96	85.00	170.00	.80	2.00	57.125	47.125	54	43.375
San Francisco	47.00	.29L	.522L	2.088L	5.67L	103.50	172.50	.45	2.20	53.69	47	50	39
Seattle	48.50	.4225	.63†	3.06†	7.55†	85.00	200.00†	.50†	1.80	65.80	57.25	66.75	56.30

*b. & s. class B and heavier, C/L lots, 200 tons and over. Burlington, N. J. (base) $43.00. Gas pipe and class A, $3 per ton additional, 4-in., $3 per ton additional. 30 in. and larger usually $2 per ton less.

#Discounts from standard list for delivery from warehouse, except Pittsburgh prices are f.o.b. mill. Base price $200 per net ton. List prices per ft.: ½ in., 8½c; ¾ in. 11½c.; 1 in., 17c.; 2 in., 37c.; 2½ in., 58½c.; 3 in., 76½c.; 4 in., $1.09; 6 in., $1.92. k2½ to 3 in. aResale price to consumer, car load lots shipped from mill.

‡f.o.b. †Less 30% in C/L Lots. c Concrete Culvert pipe reinforced. dDouble strength. lList to dealers. L list. tSales & municipal taxes extra

LUMBER AND TIMBER—PER M FT. B.M., CARLOAD LOTS F.O.B.

		SHORT LEAF YELLOW PINE AND DOUGLAS FIR All S. L. Y. P. is No. 2 common or better and for No. 1 N. C. Box. All Fir planks No. 2 common; Fir timber is No. 1 common. Lengths up to 20 ft.									**LONG LEAF Y. P.** Merchantable grade up to 20 ft.	
		1x6,^{S}	1x8,^{S}	2x4,^{S}	2x6,^{S}	2x8,^{S}	2x10,^{S}	3x12,^{R}	6x12,^{R}	12x12,^{R}	2x12,^{R}	12x12,^{R}
Atlanta	Southern Pine	$16.00	$16.00	$16.00	$18.00	$18.00	$19.50	$24.00	.....	.....	$38.50	$38.50
Baltimore	Southern Pine	23.50	24.50	27.50	26.50	27.00	29.00	37.50	.....	.....	55.00	57.00
	Douglas Fir	29.50	29.50	28.50	29.50	28.50	28.50	30.50	$29.00	$29.00	.....	.....
Birmingham	Southern Pine	21.00	21.00	22.00	20.00	21.00	23.00	30.00	30.00	28.00	.....	.....
	Long Leaf	22.00	22.00	25.00	24.00	25.00	28.00	45.00	.....	.....	45.00	45.00
Boston	Roofers N.C. Pine	28.00	28.00	33.00†	35.00†	49.00†	60.00†	73.00†	.....	.....	73.00	73.00
	Douglas Fir	27.00	27.00	31.00	33.00	35.00	37.00	41.00	40.00	41.00	.....	.....
Chicago	Southern Pine	29.00	29.50	33.50	32.50	32.50	35.50	45.50	.....	.....	54.50	54.50
Cincinnati	Southern Pine	25.50	25.50	26.50	26.00	27.00	29.00	43.00	.....	.....	45.50	55.50
	Douglas Fir	.....	.....		.....	.....		43.75	43.75	43.75	.....	.....
Cleveland	Southern Pine	30.00	32.00	38.00	32.00	36.00	38.00	49.00	44.00f	46.00f	52.00	64.00§
Dallas	Southern Pine	d28.35	30.60	31.50	29.80	30.15	30.15	50.00	82.50	66.50	42.75	67.95
Denver	Douglas Fir	42.00	42.00*m	43.00*	43.00*	43.00*	47.00*	52.00*	56.00*	61.50	.....	.....
Detroit	Southern Pine	27.50	27.50	27.50	27.50	27.50	31.25	38.50	.....	.....	32.50	44.50
	Douglas Fir	.....	.....	.....	.....	.....	.....	43.50	43.50	44.00	.....	.....
Kansas City	Southern Pine	26.50	27.50	26.50	25.50	26.50	28.00	37.00	.....	.....	55.00	62.50
	Douglas Fir	28.50	29.50	30.00	27.50	28.00	30.00	35.00	37.00	37.00	.....	..
Los Angeles	Douglas Fir	31.00y	31.00y	31.00y	31.00y	31.00y	31.00y	38.00y	48.00y	41.00y	.....	.....
Minneapolis c	Western Pine	45.00*	51.50*	50.00*	41.00*	41.00*	42.00*	.....	.....	.....	58.00*b	.....
	Douglas Fir	42.00*	42.00*	43.00*	43.00*	43.00*	43.00*	55.00*	59.50*	59.50*	54.00	.....
Montreal	Spruce	31.00	32.00	28.00	27.00	30.50	39.25	41.00	.....	.....	.....	.....
	Douglas Fir	47.50	53.00	42.50	44.00	44.00	45.75	52.00	52.00	52.00	43.25	.....
New Orleans	Southern Pine	23.50	25.50	28.00	25.00	27.00	25.50	38.00	.....	.....	44.00	54.00
	Douglas Fir	28.50	30.00	33.00	29.00	32.00	33.50	37.00	.....	.....	.....	.....
New York	Southern Pine	33.00*	34.00*	33.00*	34.00*	34.00*	38.00*	45.00*	.....	.....	72.50*	72.50*
	Douglas Fir	.....	.....	37.00*	37.00*	35.00*	38.00*	40.00*	43.00*	43.00*	.....	.....
Philadelphia	Southern Pine	30.00	31.00	32.00	32.00	33.00	34.00	36.00	.....	.....	36.00	42.00
	Douglas Fir	45.00	45.00	45.00	45.00	45.00	45.00	45.00	42.00	42.00	.....	.....
Pittsburgh	Southern Pine	30.50	31.90	37.65	33.75	36.52	37.50	47.75	.....	.....	57.50	57.50
	Douglas Fir	38.52	39.70	39.70	38.55	39.10	39.38	53.25	53.25	53.25	.....	.....
St. Louis	Southern Pine	28.50*x	29.50*x	30.00*x	29.00*x	30.00*x	31.00*x	46.00*x	.....	.....	.....	.....
	Douglas Fir	.....	.....	.....	.....	.....	.....		55.00*x	55.00*x	75.00	70.00
San Francisco	Douglas Fir	20.50t	20.50t	21.00t	21.00t	21.50t	22.50t	22.00t	25.00t	25.00t	.....	.....
Seattle	Douglas Fir	15.50	15.50	15.50	14.50	14.50	14.50	21.00	20.00	20.00	.....	.....

*Delivered. †Spruce. ‡Long-leafed Y.P. §Norway Pine. bNorthern Pine. mNative. tAt ship's tackle. yYard prices. d10% Disc. taken off. x5M ft. or less. ^{S}S4S. ^{R}Rgh. cContractors in Minneapolis and St. Paul get 15% disc. from list. f Douglas Fir.

CURRENT BUILDING AND CONSTRUCTION WAGE RATES PER HOUR

	Bricklayers	Carpenters	Structural Iron Workers	Hoisting Engineers	Plasterers	Common Labor Building	Common Labor Heavy Const.
Atlanta	$0.70/1.125	$0.40/.90	$0.60/1.25	$0.75/1.50	$0.60/1.00	$0.30/.40	$0.30/.40
Baltimore	1.25	1.10	1.37½	1.25	1.25	.45	.45
Birmingham	1.00	1.00	1.25	1.125	1.00	.30/.40†	.30/.40†
Boston	1.30	.70/1.175	1.20	.90/1.425	1.375	.50/.625	.50/.70
Cincinnati	.80/1.375	.65/1.20	1.25	1.00/1.25	.80/1.375	.45	.45
Chicago	1.50	1.50	1.50	1.50	1.50	.95	.95
Cleveland	1.375	1.25	1.25	1.25	1.375	.82	.82
Dallas	.50/1.00	.50/1.00	.50/1.00	.50/1.00	.50/1.00	.25/.40	.25/.40
Denver	1.10	1.10	1.10		.....	.625	.625
Detroit	1.25	.90/1.00	1.125/1.25	1.00/1.25	1.25	.50/.75	.50/.75
Kansas City	1.325	1.125	1.25	1.25	1.325	.70	.45/.50
Los Angeles	1.10	1.10	1.125	1.125	1.25	.50	.60
Minneapolis	1.25	1.00/1.10	1.00/1.25	1.00/1.25	1.35	.60/.75	.60/.75
New Orleans	1.00	.75	1.00	1.00	1.25	.30/.40	.30/.40
New York	1.50	1.40	1.475/1.65	1.50/1.5625	1.50	.82½	.50/.80
Philadelphia	.35/1.50	.65/1.10	.70/1.375	.90/1.00	1.10/1.37	.35/50	.40/.50
Pittsburgh	1.50	1.25	1.375	1.00	1.50	.70	.50
St. Louis	1.50	1.25	1.47	1.35/1.47	1.50	.78½	.35/.78½
San Francisco	1.50	1.125	1.37½	1.12½	1.50	.875	.60
Seattle	1.00	1.00	1.25	1.25	1.25	.60	.60
Montreal	.70	.60	.80	.55	.70	.35	.40

Skilled Avg: (Bricklayers, Carpenters, Ironworkers) $1.159 Common Avg. $0.569 † PWA.